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LINER ENGINEERING REPORT

FORD MOTOR COMPANY - ALLEN PARK CLAY MINE
MID 980568711
SECTION D

Prepared by Neyer, Tiseo & Hindo, Ltd.
March, 1988
Revised June 24, 1988

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(Prepared by Neyer, Tiseo & Hindo, Ltd. - June 24, 1988)

1.0 INTRODUCTION

The following submittal, prepared by Neyer, Tiseo and Hindo, Ltd. (NTH), presents a discussion and evaluation of the proposed double liner system design intended for use in the hazardous waste disposal Cell II at Ford Motor Company's Allen Park Clay Mine Landfill. This submittal has been prepared in accordance with requirements outlined in Michigan Act 64 R.299.9505, R299.9619, R299.9620 and R299.9621, as well as 40 CFR 264.301 (a & c).

This evaluation of the double liner system by NTH is based on design information available at the time of writing and may require modification if the liner design is changed. Our evaluation has been performed according to generally accepted geotechnical engineering practices for the exclusive use of Ford Motor Company and the Michigan Department of Natural Resources (MDNR).

2.0 DESCRIPTION

2.1 Pre-Liner Construction Activities

Cell II has been excavated to an approximate elevation of 560 feet mean sea level (msl). Prior to construction of the Cell II double liner system, cutting and/or filling operations will be performed so that the Cell II base and sideslopes conform to elevations and grades given in engineering design plans. Other pre-liner construction activities include the construction of a U-shaped stabilization berm at the toe of the slope and installation of a pressure relief system at the bottom of the cell. Engineering design plans indicate that a berm will be built around the perimeter of Cell II to provide an anchor trench for the liner and the cover system. It is expected that construction of this berm will occur concurrent with the double liner system construction.

2.2 Double Liner System

The proposed double liner system intended for use in Cell II of the Allen Park Clay Mine (APCM) will consist of the following five basic elements, from the bottom upward:

A. pressure relief system

B. an 80-mil secondary flexible membrane liner (FML);

C. a leak detection system;

D. a composite liner consisting of 5 feet of compacted clay overlain by an 80-mil primary flexible membrane liner; and

E. a leachate collection system.

Except in areas where sideslope backfilling to achieve design grades will occur, the double liner system will generally rest upon the native silty clay subsoils. The eastern sideslope of the cell (grid direction) will be founded on the interim cover and underlying compacted fill material of previously closed Cell I.

3.0 NATURAL SOILS/FOUNDATION

3.1 On-Site Soil Conditions

Geologic conditions at APCM are characterized by approximately 100 to 110 feet of clayey soils overlying the Dundee Limestone (Mozola, 1969). Soil investigations performed by Michigan Testing Engineers, Inc. (MTE) and NTH, as well as on-site clay mining operations, have confirmed this extensive clay deposit. An MTE hydrogeological report dated November 24, 1981, indicated that the native silty clay deposit extended to depths comparable to those found in other investigations.

A subsoil investigation performed by NTH in January, 1985, substantiated data obtained during earlier studies. The body of data generated during this investigation is presented in Appendix I. Based on this data, the generalized subsurface profile beneath Cell II of APCM is characterized by a deposit of gray silty clay that is underlain by a unit of hard clayey silt locally termed hardpan. Test boring data included as Figures 1 through 3 in Appendix I, indicate that the silty clay deposit extends to a depth of approximately 95 feet below the natural ground surface. As shown by vane shear test data presented on Figures 6 through 10 of Appendix I, the consistency of the clay was found to range from soft to medium in the lower 20 feet of the deposit. The clay is of medium consistency in the upper 75 feet of the deposit. Tabulation of soil test data is presented on Figures 4 through 5 of Appendix I. As indicated by these data, these soils were found to be moderately plastic and have a Unified Soil Classification (USC) of CL (ASTM D2487). The moisture content and dry density of selected samples of these soils ranged from 18 to 37% and 85 to 116 pounds per cubic foot, respectively.

Clay mining operations have shown that soil conditions described in MTE and NTH reports appear to be present over the entire site. During these operations, clay was excavated to a depth of approximately 45 feet over a large portion of the site.

The NTH investigation revealed that groundwater is present in deeply buried granular and/or limestone deposits beneath the site. These water bearing strata are under significant artesian pressure, resulting in piezometric levels at approximately elevation 605. This piezometric level is above the existing ground surface around Cell II, effectively resulting in an upward hydraulic gradient over the entire site.

3.2 Stability [299.9505 (1)(d)(iii,v), 299.9621 (1)(b)]

The foundation stability of the proposed design slope configuration of Cell II has been evaluated with respect to two possible modes of failure: deep failure with the failure surface tangent to the hardpan layer and shallow failure through the toe of the slope.

Using the computer adaption of the Bishop method of slices, many different slip surfaces corresponding to the two failure modes were evaluated. The failure surfaces having the lowest factors of safety are presented on Figure 1 of Appendix II. These surfaces correspond to surfaces over which failure would most likely occur for design slope configuration and soil conditions shown in Figure 1. As shown on Figure 1, a U-shaped intermediate clay dike acting as a stabilizing toe berm for the cell sideslopes is included in the cell sideslope design. The effects of traffic surcharge load are also included. Soil conditions on which the analyses were based consisted of undrained shear strength values of 920 and 690 psf for the underlying medium and medium to soft clay, respectively. These values are based on vane shear data presented on Figures 6 through 10 of Appendix I. Acceptable factors of safety (not less than 1.2) were obtained for the slip surfaces shown on Figure 1. As shown on Figure 1, the analysis accounts for truck traffic along the top of the slope. The analysis assumes no storage of construction materials or stockpiling of soil backfill is allowed within 100 feet of the top of the slope.

Calculations included in Figures 2 through 4 of Appendix II show that there is a potential for base instability due to artesian pressures in the underlying aquifer. Therefore, it is intended that a program of cell base preparation be undertaken in accordance to specifications outlined in the Construction Quality Assurance Plan, (CQAP), dated January 21, 1988, and revised June 24, 1988.

3.3 Settlement [299.9505 (1)(d)(i)]

Compressibility characteristics of the soils beneath the landfill cell were examined and are included in Figure 11 of Appendix I. Settlement calculations based on these compressibility characteristics are presented in Appendix II, Figures 5 through 10. As shown on Figure 10, the maximum anticipated settlement after liner installation and filling of the cell is approximately 2 feet. Since localized concentrated loadings are not expected, settlement is expected to occur over a large area. Hence, localized differential settlements beneath the base of the cell are expected to be minimal. Therefore, the anticipated settlement should not adversely affect the liner integrity.

Some consolidation is expected to occur within the fill materials underlying the western side of Cell I as well as within the native clay beneath the other cell sideslopes. Settlement resulting from this consolidation is similarly not expected to adversely affect the integrity of the liner system.

3.4 Bearing Capacity for Manhole [299.9505(91)(d)(ii)]

In 1985, an analysis was performed to evaluate the ability of the subgrade to support a reinforced concrete manhole. This analysis is presented in Figures 11 and 12 of Appendix II. The HDPE manhole incorporated into the latest liner design will be "lighter" than the concrete manhole. Therefore, subsoils are expected to provide adequate support for the proposed manhole.

3.5 Varying Groundwater Conditions [299.9505(1)(d)(iv)]

Varying groundwater conditions are unlikely to have a significant effect on the performance of the liner system once Cell II is constructed and filled. A rise in the piezometric level in the aquifer during construction and the early stages of filling would increase the potential for basal instability. For this reason, preparation of the cell base and initial filling should be performed as expeditiously as possible.

Changes in the pore pressures in the cohesive deposits brought on by a change in groundwater conditions would affect the settlement analysis. However, a large increase in either total or differential settlement above the maximum estimates presented above would not be expected unless excessive dewatering of the underlying aquifer is undertaken.

4.0 PRESSURE RELIEF SYSTEM [264.301(a)(1)(i)]

4.1 Purpose

As indicated in the previous section, an upward hydraulic gradient exists at the APCM site. Upward seepage could ultimately result in an accumulation of water beneath the liner on the base or the side slopes. Such an accumulation could in turn result in a loss of friction along the sideslopes or in damage to the compacted clay liner along the base. To mitigate possible liner instabilities related to seepage, a pressure relief system has been designed to intercept and collect water that might accumulate beneath the secondary FML.

4.2 Elements and Design

In general, the pressure relief system consists of a series of wick drains spaced at 50-foot intervals along the cell base and at 25-foot intervals along the cell walls, and located all along the toe of the slope. These drains lead to 4-inch diameter HDPE collector pipes which in turn allow the captured water to flow to a sump for eventual disposal. The general layout of this system is shown on Cell Engineering Design Plans. Calculations on which the design of the pressure relief system is based are presented on Figures 13 through 22 of Appendix II.

4.3 Transmissivity of Wick Drains

As shown on Figure 14, wick drains will consist of 5-foot wide strips of geosynthetic drainage net placed directly beneath the 80-mil FML. A sheet of geosynthetic filter fabric will exist between the drainage net and the underlying subbase clay to prevent the migration of clay particles into the drains. The transmissivity of the drainage wick was evaluated to determine if the wicks could efficiently transmit seepage. Based on calculations presented in Figures 14, 15, and 22C through 22E of Appendix II, drainage nets having a transmissivity on the order of 10^{-5} m^2/s, such as the TENSAR(TM) DNI or equivalent, would meet this requirement. Manufacturers' hydraulic transmissivity test results for various types of drainage nets are shown on Figure 16 for information.

4.4 Filter Requirements

Filter criteria for the interface between the filter fabric and clay subbase have been evaluated. This evaluation is presented in Figure 17 of Appendix II. Filter criteria were used to evaluate the potential for clay particles to enter and clog the filter fabric. Based on our evaluation, any fabric having an equivalent opening size greater than 0.149 mm and less than 0.211 mm should be suitable. Physical properties of two brands of filter fabric, including the equivalent opening size, are listed on Figures 18 and 19 of Appendix II. Both of these satisfy the required filter criteria.

4.5 Pipe Minimum Design Slope and Perforations

It is anticipated that 4-inch diameter HDPE pipes (SDR 21) will be sloped at a 1% grade, consistent with the grade of the Cell II base. Based on calculations presented in Figure 20, Appendix II, 4-inch diameter pipes sloped at a 1% grade will effectively transmit anticipated design flows.

It is anticipated that perforations within the HDPE pipe will consist of 2 rows of 1/4-inch diameter circular holes spaced at a 60° angle. Based on calculations presented in Figures 20 and 21, Appendix II, perforations of this size and spacing will accommodate design flows preventing an excess pressure gradient from developing at the entrance of the pipe.

4.6 Pipe Strength [299.9505 (1)(e)(ii)(F), 40 CFR 264.301 (a)(2)(i)(B)]

Calculations performed to determine if the strength of 4-inch diameter HDPE pipe will sustain the load of overlying material are presented in Appendix II, Figures 22A and through 22B. As shown in Figure 22B, the weight of the overlying refuse and the granular drainage blanket will result in a pipe deflection of 1.9%. This value is less than the maximum allowable deflection of 7.5%, recommended by a manufacturer of HDPE pipe. Figure 40 suggests that equipment having a ground contact pressure greater than 41.5 psi should not be used within a distance of five feet above the upper surface of the granular blanket.

5.0 CLAY SOURCES USED IN CELL AND LINER CONSTRUCTION [299.9620(2), 299.9505 (1)(b), 40 CFR 264.301 (a)(1)(i)]

Compacted clay will be placed on the base and sideslopes of Cell II, used in berm construction and in the construction of the 5-foot compacted clay liner. Presently, two sources of clay have been specified for use. These sources include native silty clay obtained from on-site clay mining operations and silty clay obtained from the I-696 highway construction project.

5.1 Native On-Site Silty Clay - Test Data

Laboratory testing data generated for the native on-site silty clay has been compiled and is presented on Table 1, Characteristics of Clay Sources; Table 2, Results of Laboratory Strength Testing For Native On-site Clay; and Table 4, Summary of Permeability Test Results. These tests were performed on bag and Shelby tube samples obtained during the summer of 1986 and on bag samples obtained in the spring of 1987.

Table 2 contains the results of a series of unconfined compressive strength tests performed on soil samples from native on-site clay prepared at varying moisture contents and densities. Undrained shear strength values shown on Table 2 are equal to 1/2 of the unconfined compressive strength value for a given sample. The results of permeability testing with water are shown on Table 4.

In addition to laboratory data generated in 1986 and 1987, laboratory testing of native silty clays was undertaken during a subsoil investigation performed by NTH in 1985. At that time, soil samples were obtained from the drilling of two deep test borings. Logs of these borings are presented in Figures 1 and 2 of Appendix I. As shown on these boring logs, liner samples and relatively undisturbed piston samples were obtained at various depths throughout the extensive unit of soft to medium gray silty clay underlying the site. Various tests (water content, density, Atterberg limits, field vane shear test and one consolidation test) were performed on selected samples from this investigation. Results of this testing are included on Figures 4 and 5 of Appendix I, Tabulation of Test Data.

The overall suitability of this material for use in the Construction of Cell II is discussed in Section 14.0, Constructability of the Cell and Double-Liner System. The suitability of this material for use in construction of various components of Cell II is discussed in Sections 6.1, 7.2, and 9.2.

5.2 I-696 Clay - Test Data

Laboratory testing data generated for the I-696 clay has been compiled and is presented on Table 1, Characteristics of Clay Sources, Table 3, Results of Laboratory Strength Testing for I-696 Clay, and Table 4, Summary of Permeability Test Results. Tests were performed on bag samples obtained between the summer and spring of 1987.

Table 1 contains the results of Modified Proctor, grain size distribution and Atterberg limit determinations performed on samples of I-696 clay. Table 3 contains the results of a series of unconfined compressive strength tests performed on I-696 samples prepared at varying moisture contents and densities. Undrained shear strength values shown on Table 3 are equal to 1/2 the unconfined compressive strength value for a given sample. Permeability testing results for this material are presented on Table 4.

The overall suitability of this material for use in the Construction of Cell II is discussed in Section 14.0, Constructability of the Cell and Double Lined System. The suitability of this material for use in construction of various components of Cell II is discussed in Sections 6.1, 7.2, and 9.2.

5.3 Index Properties of Native On-Site Clay

As indicated in above sections, properties of native on-site clay have been compiled and are included in Table 1, Characteristics of Clay Sources. Based on properties presented in Table 1, all samples may be classified as CL soils under the USC system (ASTM D2487). The percent by weight of sample material passing the #200 sieve ranges from 71 to 99.6%. The liquid limit for this material ranges from 21 to 34, while the plasticity index ranges from 8 to 16.

As shown on Table 1, the maximum dry density of native on-site clay as determined by the Modified Proctor Test (ASTM D557) ranges from 113.6 to 129.7 pounds per cubic foot. The optimum moisture ranges from 9.2 to 14.9%. These moisture content values are significantly lower than the natural values of this material, which range between 12.4 and 33.1% as shown on Figures 4 and 5 of Appendix I.

5.4 Shear Strength of Native On-Site Clay

As shown on Table 2, Results of Laboratory Strength Testing for Native On-Site Clay, the undrained shear strength was determined for a number of samples prepared at moisture contents of approximately +3%, +5%, and +9% above optimum and at varying degrees of compaction. As shown on this table, samples prepared at moisture contents approaching natural water content values (approximately 18%) tended to have a very soft to medium consistency and generally could not be compacted to a dry density of more than 88% of the maximum dry density for the material. On the other hand, samples prepared at moisture contents closer to optimum tended to have a very stiff to hard consistency and could be compacted to dry densities of more than 90% of the maximum dry density for that material.

5.5 Permeability of Native On-Site Clay

As shown on Table 4, Summary of Permeability Test Results, permeability testing was performed on three sources of the native on-site clay. Samples tested generally were compacted at either 90% or 95% of the maximum dry density for that material. Moisture contents of the samples ranged from -2% to +5% of the optimum value. The results of 15 permeability tests are included in this table.

In general, permeability test results were somewhat variable. In most cases, samples tested at 95% compaction and at moisture contents of 5% above optimum consistently tended to yield acceptable coefficients of permeability, i.e., slightly below 1 x 10^{-7} cm/sec. Samples prepared at 95% compaction and at moisture contents of 2% below optimum, as well as all samples prepared at 90% compaction, generally yielded coefficients of permeability unacceptable for the compacted clay portion of the double liner system.

5.6 Index Properties Of I-696 Clay

As indicated in preceding sections, properties of I-696 clay have been compiled and are included on Table 1, Characteristics of Clay Sources. Based on properties presented in Table 1, all samples may be classified as CL soils under the USC system (ASTM D2487). The percent by weight of sample materials passing through the #200 sieve ranges from 64.4 to 72.3%. The liquid limit for this material ranges from 25 to 29 while the plasticity index ranges from 9 to 14.

As shown on Table 1, the maximum dry density of I-696 clay as determined by the Modified Proctor Test (ASTM D557) ranges from 120.2 to 132.6 pounds per cubic foot. The optimum moisture ranges from 8 to 13.6%.

5.7 Shear Strength of I-696 Clay

As shown on Table 3, Results of Laboratory Strength Testing for I-696 Clay, the undrained shear strength was determined for a number of samples prepared at moisture contents of approximately -2% below optimum, at optimum, +2% and +5% above optimum, and at varying percents of compactive effort. Based on the results of strength testing, samples prepared at moisture contents of +5% optimum tended to have relatively low undrained shear strengths which corresponded to soft to medium consistencies. Generally, these samples could not be compacted to dry densities greater than 89% of the maximum dry density for that material. Samples prepared at -2% below optimum tended to have higher undrained shear strengths which corresponded to stiff to very stiff consistencies. Generally, a compaction of 92% of the maximum dry density could be obtained in these samples. Finally, samples compacted at optimum moisture content or 2% above optimum tended to also have high unconfined strength which corresponded to consistencies ranging from hard to very stiff. Samples prepared at optimum moisture content reached high undrained shear strengths and compaction levels as high as 95.7% of the maximum dry density.

5.8 Permeability of I-696 Clay

As shown on Table 4, Summary of Permeability Test Results, permeability testing was performed on four sources of the I-696 clay. Samples tested generally were completed at either 90% or 95% of the maximum dry density for that material. Moisture contents of the samples ranged from -2% to +5% of the optimum value. The results of 16 tests are included in this table.

Permeability test results for the I-696 clay samples were less variable than those for the native on-site clay samples. In all cases, permeability coefficients were less than 1 x 10^{-7} cm/sec, the maximum acceptable value for liner material. As shown on Table 4, one sample initially yielded a permeability coefficient of 2.3 x 10^{-7} cm/sec. However, this test was later rerun on another sample of the same material and prepared in the same manner and yielded a permeability coefficient of 0.77 x 10^{-7} cm/sec.

5.9 Permeability of Source Materials with Respect to Leachate [299.9505 (1)(b)(vii)]

Permeability testing using a synthetic leachate as a permeant, was conducted on I-696 clay as well as native on-site material. The test was performed in a triaxial cell on samples prepared at optimum moisture content and 90% compaction. Each sample was permeated with water and allowed to equilibrate prior to the introduction of leachate. The leachate used in the test was synthesized by spiking leachate generated in Cell I with constituents expected to be placed in Cell II.

Details on testing methodology as well as the results of the testing are discussed in Appendix IV. Based on these results, permeation of both the I-696 and the native soil samples with leachate did not appear to adversely affect the permeability of the clay.

5.10 Selection & Testing of Other Material Sources

Characteristics of clay sources intended for use in construction have been described in the preceding paragraphs. Other material sources may eventually be considered for use. Material intended for use in the compacted clay liner will have the following characteristics:

A. A Unified Soil Classification (USC) of CL as determined by the provisions of ASTM Standard D2487.

B. More than 25% of the soil particles will be less than 5 microns in size.

C. Are capable of being compacted to achieve a permeability coefficient (after compaction) of not more than 1 x 10^{-7} cm/sec.

D. Are capable of being compacted to achieve an undrained shear strength of at least 2500 psf for those areas identified in Plate 1.

Permeability, grain size and soil classification criteria are based on the requirements of rules 299.9505(1)(b)(vi) and 299.9620(2)(a,b,d) of Act 64. Shear strength criteria is derived from a slope stability analyses performed by NTH, which indicate that a minimum undrained shear strength of 2500 psf is needed over most of the slope. This value is expected to provide an adequate factor of safety against slippage of the compacted clay along the underlying geotextiles for the existing design slopes.

The rationale for selection of this shear strength parameter is discussed in greater detail in Section 9.0, Compacted Clay in the Primary Liner.

Potential borrow sources of CL material will be sampled and tested to determine if the source is suitable for use in the Cell II liner system or in other areas of cell construction. Such testing will include determination of grain size distribution and Atterberg limits. Additional testing of the potential borrow source samples, after compaction in the laboratory, will include moisture-density relationship (Modified Proctor), permeability, and unconfined compressive strengths. Permeability testing using a synthesized leachate permeant similar in composition to the anticipated cell leachate will also be performed. Testing will be performed, where appropriate, according to ASTM standard methods referenced in R299.9505(1)(b).

6.0 SUBBASE AND STABILIZATION BERM CONSTRUCTION

The base of Cell II has been excavated to an approximate elevation of 560 feet msl. Prior to construction, compacted clay fill will be placed in the existing cell base to reach design grades and a stabilization berm will be built along the base of the cell as shown on the drawings.

Permeability and shear strength criteria have been established for source materials intended for use in construction of the U-shaped stabilization berm and clay subbase. Permeability criteria are based on Act 64 requirements; shear strength criteria are based on slope stability analyses discussed in Section 9.0.

Moisture/density specifications that will enable the soil fill source to meet permeability and shear strength requirements are based on laboratory test data presented in Tables 2 through 4. These requirements have been summarized and are presented on Plate 1, Cell II Liner System Construction Requirements. Plate 1 specifies the shear strength and permeability which apply to various portions of the cell and liner as well as moisture/density specifications needed to meet these criteria.

6.1 Suitability of Source Materials

As previously indicated and as presented on Plate 1, a shear strength of 2500 psf is required for clay used to construct the U-shaped stabilization berms. This shear strength can be achieved in either the native on-site clay or the I-696 clay. However, laboratory test data for the I-696 clay, presented on Table 3, Results of Laboratory Strength Testing For I-696 Clay, suggest that the range in moisture content over which the clay may be placed must be restricted in order to achieve the minimum required shear strength of 2500 psf. As shown on Plate 1, it is recommended that I-696 clay used in construction of the stabilization berms be placed at 90% compaction and moisture contents ranging between 2% below to 3% above optimum. Since the shear strength requirements for the compacted clay subbase are much lower (500 psf - see Plate 1), I-696 clay used for construction in this area may be placed at 90% compaction and moisture contents ranging between 2% below to 5% above optimum.

As shown on Table 2, Results of Laboratory Strength Testing For Native On-Site Clay, the native on-site clay meets shear strength requirements for the stabilization berm when compacted at moisture contents below +5% of optimum. Therefore, this material may be placed at 90% compaction and moisture contents ranging between 2% below to 5% above optimum, as indicated on Plate 1.

As shown on Plate 1, minimal shear strength and permeability restrictions apply to the construction of the Cell II subbase. Therefore, on site clay may be placed at 90% compaction and moisture contents ranging from 2% below optimum to 5% above optimum and still meet shear strength and permeability requirements of 500 psf and 1×10^{-6} cm/s, respectively. Accordingly, both the native on-site clay and I-696 clay may be used in the preparation of the Cell II subbase.

6.2 Construction Requirements

Construction of the subbase and U-shaped stabilization berm will proceed in accordance with specifications outlined in the Construction Quality Assurance Plan dated June 24, 1988. General construction specifications, testing type and frequency and responsibilities of various organizations involved in construction are included in this document.

7.0 SIDESLOPE AND PERIMETER BERM CONSTRUCTION

7.1 Filled Slopes and Side Berms

As shown on engineering design plans, fill will be placed in some areas along the Cell II sideslopes in order to achieve design grades in these locations. In addition, a side berm (perimeter berm) will be constructed around the perimeter of Cell II. Permeability and shear strength criteria which apply to soil used in berm construction and sideslope preparation are shown on Plate 1 as well as moisture/density specifications need to meet these criteria. Construction of the sideslope and side berm will proceed in accordance with specifications outlined in the Construction Quality Assurance Plan.

7.2 Suitability of Construction Materials

As shown on Plate 1, a shear strength of 2500 psf is required for clay placed along Cell II sideslopes and in the side berm construction. Based on laboratory test data presented on Table 3, it is recommended that I-696 clay used in constructions of these portions of Cell II be placed at 90% compaction and moisture contents ranging between 2% below to 3% above optimum.

Laboratory test data presented in Table 4 indicates that the native on-site clay may be placed at 90% compaction and moisture contents ranging between 2% below to 5% above.

7.3 Construction Requirements

General construction specifications, testing type and frequency and responsibilities of various organizations involved in construction are included in the Construction Quality Assurance Plan document.

7.4 Cut Slopes

As shown on engineering design drawings, some cutting of sideslopes will be performed in order to achieve design grades. General construction specifications, testing type and frequency and responsibilities of various organizations involved in construction are included in the Construction Quality Assurance Plan document.

8.0 LEAK DETECTION AND SECONDARY LINER SYSTEM

8.1 Description and Physical Properties [299.9622(1), 299.9505(1)(e)(ii,iii)]

Multiple layers of geosynthetics composing the leak detection system and the secondary FML comprise the lowermost containment unit of the double liner system. They are shown on the engineering design drawings prepared by MCI. The leak detection system underlying the 5-foot compacted soil layer in Cell II consists of a combination of geotextile drainage nets, filter fabrics, and HDPE collection pipes leading toward four collection sumps. This combination of material facilitates the detection of potential leaks through the clay layer.

In general, two layers of HDPE drainage net will be placed along the base of Cell II. A single layer of thicker HDPE drainage net will be placed along the sideslopes. All drainage net layers will be placed in a manner to maximize the space between individual drainage sheets and will be covered with filter fabric. Six-inch diameter SDR 7.3 HDPE perforated collection pipe with capped ends will be placed in the collection point with five layers of drainage net. This pipe will be joined to six-inch diameter SDR 7.3 nonperforated pipe which will pass through the secondary liner to a storage sump located near the cell boundary. Another six-inch nonperforated pipe will provide access from the sump to the surface. The secondary FML will be sealed at the point where the secondary leak detection piping passes through. The primary FML will not be perforated at any point by the six-inch collection piping. A 80-mil HDPE FML will underlie the leak detection drainage net system as the secondary liner system. The properties of this materials are listed in Standard 54 (NSF. 1985).

8.2 Chemical Compatibility [299.9505(1)(c)(iii), 299.9505 (1)(e)(ii)(E)]

Material properties of the secondary FML and of geosynthetics of the leak detection system will be evaluated following the selection of a manufacturer of these materials. Such properties will be used to evaluate chemical compatibility between geosynthetics and the landfill environment. Any additional information necessary to evaluate chemical compatibility as required by R299.9505 (1)(c)(iii) and 299.9505 (1)(e)(E) will be obtained at this time.

8.3 Damage to Geosynthetics During Placement of Compacted Clay [299.9505 (1)(e)(ii)(F)]

Damage to geosynthetics during placement and compaction of the clay portion of the primary liner is a consideration that was evaluated by Wayne Disposal, Inc. (WDI) via construction of a test fill prior to construction of Master Cell VI at the WDI Landfill Site No. 2. The purpose of the test fill was to evaluate the stability of a compacted clay layer on geosynthetics and to investigate the potential for damage to geotextiles during clay placement and compaction. The liner system used by WDI in Master Cell VI is essentially identical to that proposed for use in Cell II as described herein. Therefore, findings for the test fill that relate to damage to geosynthetics during compacted clay placement are applicable for evaluation of the double liner system in Cell II. These findings were described in a submittal prepared by NTH entitled "Findings for the Test Fill" and dated July 31, 1986. This document is contained within Appendix III of this submittal.

As indicated by the test fill document, several pertinent conclusions were drawn following the completion of the test fill. The most important with respect to the leak detection system were that the earth moving equipment and methods used to construct the test fill did not appear to damage the geosynthetics or cause excessive stresses if proper precautions were taken. Furthermore, the drainage and leak detection system was not rendered ineffective by the clay compaction operations.

8.4 Installation and Operation Stresses [299.9505 (1)(e)(ii)(F)]

Drainage nets of the leak detection system and the secondary FML are expected to experience tensile stresses during placement along the sideslopes. Calculations determining the magnitude of these stresses are presented in Appendix II, Figures 23 and 24. As shown in these calculations, stresses within the drainage net and 80-mil FML during installation will be expected to be 3.2 lb/in and 1.3 lb/in, respectively. These are well below the tensile standard of 140 lb/in listed in Standard 54 for 80-mil HDPE or the tensile specification of 53 lb/in provided by manufacturers of the PN-3000, which is a typical drainage net considered for use in the liner.

The secondary FML may be subjected to puncture stresses due to objects that may be present on the native clay soils underlying the secondary FML. A D-6 Caterpillar LGP dozer will exert a stress on the FML of approximately 7 psi during compaction of the overlying 5-foot of compacted clay liner. This stress is less than the 650 psi hydrostatic resistance specification provided by various HDPE manufacturers for 80-mil HDPE. Consequently, a dozer of this size is not expected to damage the FML during compaction if operated with care. Puncture also will be avoided through careful preparation and inspection of the subgrade prior to placement of the FML.

Elongation of the geomembranes of the leak detection/secondary liner system may occur due to potential foundation settlement following the placement of waste. As shown on calculations in Appendix II, Figure 25, these strains are not expected to exceed the maximum allowable limit of 10%.

8.5 Transmissivity of Drainage Nets Under Load

The minimum transmissivity of geosynthetic drainage layers required to facilitate flow for a conservative estimate of fluid in the leak detection system has been determined. As indicated in calculations presented in Appendix II, Figures 26 and 27, transmissivity within drainage nets placed beneath the cell base and sideslopes will be approximately 13 cm^2/sec, a value considerably greater than transmissivity of 0.46 cm^2/sec required to accommodate design flows in these areas.

8.6 Construction Requirements [299.9505 (1)(c)(i), 299.9621 (1)(d)]

General construction specifications, testing type and frequency and responsibilities of various organizations involved in construction are included in the Construction Quality Assurance Plan document.

8.7 Pipe Strength [299.9505 (1)(e)(ii)(F), 40 CFR 264.301 (a)(2)(i)(B)]

Calculations performed to determine if the strength of 6-inch diameter HDPE pipe will sustain the load of overlying material are presented in Appendix II, Figures 36 and through 39. As shown in Figure 39, the weight of the landfill cover, overlying refuse, and the granular drainage blanket will result in a pipe deflection of 2.0% This value is equal to the maximum allowable deflection of 2.0%, recommended by a manufacturer of HDPE pipe. Figure 40 suggests that equipment having a ground contact pressure greater than 41.5 psi should not be used within a distance of five feet above the upper surface of the granular blanket.

9.0 COMPACTED CLAY IN PRIMARY LINER

A 5-foot layer of compacted clay will overlie the secondary FML and leak detection system. The stability of this compacted clay layer has been evaluated to determine minimum shear strengths required in the clay.

9.1 Sliding of the Sideslope Liner

The potential for sliding of the double liner system along the cell sideslopes has been evaluated. This evaluation was performed using a computer adaptation of the Modified Bishop Method of Slices. In general, the stability of the compacted clay with respect to sliding is dependent upon the shearing resistance of the 5-foot compacted clay layer, the friction characteristics of the double liner system and the geometry of the slope. The design slope configuration on which this analysis is based is presented in Figure 28, Appendix II. The results of this analysis are presented graphically on Figure 29 of Appendix II.

This analysis was based on an assumption that friction along the sideslopes was equal to zero. Theoretically, this situation could occur if seepage was allowed to accumulate beneath the sideslopes. The result of this analysis indicated that the stability of the double liner system could be maintained with an acceptable factor of safety (greater than 1.2) if the compacted clay layer has a minimum shear strength of 2500 psf. Consequently, it is recommended that compacted clay placed along cell sideslopes, at the toe of the slope and on the outside face of the U-shaped stabilization berm have a shear strength of 2500 psf. This shear strength requirement also applies to fill placed along sideslopes prior to construction of the liner and to material used in berm construction. It should be noted that accumulation of water and subsequent loss of friction beneath the double liner system is unlikely due to the operation of the seepage collection system. As discussed in Section 4.2, the seepage collection system consists of wick drains spaced at 50-foot on center on the cell base, and at 25-foot on center on the cell sideslopes.

9.2 Suitability of Construction Materials

As shown on Plate 1, shear strength and permeability requirements of 2500 psf and 1 x 10^{-7} cm/sec, respectively, apply to material used in construction of the clay liner. Both the native on-site clay and the I-696 clay may be used in the construction of the 5-foot compacted clay liner. However, restrictions must be placed on moisture-density requirements for both materials so that the above shear strength and permeability requirements can be met.

As indicated by laboratory test data presented in Table 3, the range in moisture content over which the I-696 clay may be placed must be restricted in order to achieve the minimum required shear strength of 2500 psf. It is recommended that I-696 clay used in construction of the 5-foot compacted clay liner be placed at 90% compaction and moisture contents ranging between 2% below to 3% above optimum, as shown on Plate 1.

It should be noted that this restriction applies only to the portion of the liner placed along cell sideslopes and on the outside face of the U-shaped stabilization berm. Clay used in constructing the portion of the liner along the inner face of the U-shaped stabilization berm and along the base may be placed at 90% compaction and moisture contents ranging from 2% below to 5% above optimum moisture content, as shown on Plate 1.

As shown on Table 4, Summary of Permeability Test Results, the permeability of the native on-site clay exceeds 1×10^{-7} cm/sec in samples prepared at 90% compaction and in samples prepared at 95% compaction with corresponding moisture contents that fell below optimum. Since the coefficient of permeability for compacted clay used in construction of the 5-foot clay liner cannot exceed 1×10^{-7} cm/sec, it is recommended that the native on-site clay placed in all portions of the 5-foot compacted clay liner be compacted to 95% of the maximum dry density and moisture contents above optimum, as shown on Plate 1.

9.3 Construction Requirements [299..9620(2)(c), 299.9505 (1)(b)(viii), 299.9621 (1)(c)]

General construction specifications, testing type and frequency and responsibilities of various organizations involved in construction are in the Construction Quality Assurance Plan document.

10.0 PRIMARY FLEXIBLE MEMBRANE LINER [299.9505(1)(c), 40 CFR 264.301 (a)(1)(i)]

10.1 Physical Properties [299.9505 (1)(c)(ii)]

The FML overlying the 5-foot compacted clay liner in Cell II will consist of 80-mil HDPE. Material property data for HDPE is provided by the manufacturer of this material. Presently, a manufacturer of 80-mil HDPE has not been selected. However, at a minimum the FML will have material properties listed in Standard

Number 54: Flexible Membrane Liners (NSF 1985) for this material. After an HDPE manufacturer has been selected, material property data will be submitted.

10.2 Exposure and Property Retention [299.9505 (1)(c)(i), 299.9505 (1)(c)(iii)]

Chemical property data is generally supplied by the FML manufacturer. Upon selection of the manufacturer, chemical property data will be obtained and evaluated in order to determine chemical compatibility of the 80-mil HDPE with the landfill environment as required by Rule 299.9505 (1)(c)(iii). Any additional chemical data needed to perform this assessment will be obtained at this time.

Measures will be undertaken to avoid exposure of the FML to adverse climatic conditions prior to and following installation of the FML. The FML will be protected from direct heat and sunlight during storage. FML installation will not occur below an ambient temperature of 1°C (34°F), above an ambient temperature of 35°C (95°F), during precipitation or high wind events. Following placement of the FML along the base of Cell II, a granular blanket will be placed to protect the FML against weathering processes. Drainage net and fabric placed along the cell sideslopes will serve as a protective medium for the FML at these locations.

10.3 Storage, Handling and Liner Installation Stresses [299.9505 (1)(c)(i)]

During construction of the double liner system, the FML will be subjected to stresses resulting from storage, handling and installation of the FML. Most stresses related to storage and handling of the FML can be minimized by careful adherence to construction requirements outlined in the Construction Quality Assurance Plan document. The FML will be stored in an area away from heavy traffic and in a location free of excess dust. As indicated in the Construction Quality Assurance Plan document, appropriate handling equipment will be used when moving rolled or folded FML from one place to another. Pulling FML panels along the ground will be minimized.

However, the FML will experience tensile stresses during installation on the sideslopes. These stresses will occur while the FML is supported only along the slope crest. Calculations performed to determine the magnitude of these stresses are presented in Appendix II, Figures 23 and 24. As shown in these calculations, the FML will experience a tensile stress of 1.3

lb/in during installation. This stress is considerably lower than the tensile stress standard of 140 lb/in that is listed in Standard 54 for 80-mil HDPE. Therefore, these stresses should not result in damage to the FML provided that the FML is securely anchored over the length of the slope crest and care is taken not to create excessive stretching by sliding sand and waste down the sideslope liner during the placement of these materials.

The FML also will be subjected to additional tensile stresses during the placement of the granular drainage blanket of the leachate collection system. These stresses will result from the type and size of construction equipment used to spread the blanket. Track-mounted vehicles such as bulldozers will impart much lower contact pressures on the liner than rubber-tire vehicles. Thus, track-mounted vehicles are suitable for placing and spreading the granular blanket for the leachate collection system.

For example, a D-6 Caterpillar LGP or similar size dozer has a ground contact pressure of approximately 7.0 psi. This value is essentially the stress that would be imposed on the FML during placement of the granular blanket. However, it is much lower than the hydrostatic resistance standard specifications of 650 psi, provided by HDPE manufacturers for 80-mil HDPE. The hydrostatic resistance test, ASTM D751-79 Method A, models a puncturing effect in a reverse fashion. Consequently, a dozer of the size described above is not expected to damage the FML during placement of the granular blanket if operated with care.

As an added measure of precaution, the full design thickness of the granular blanket should be maintained at all times during placement of the granular blanket. Where rubber-tire vehicles must be driven over the FML, a thicker layer of protective material should be used. A minimum of 24 inches of soil or waste cover over the FML is recommended for use under rubber-tire vehicles to dissipate additional stresses that these vehicles typically will impart on a granular blanket. It is emphasized that careful operation of construction equipment near the FML is always required.

10.4 Operational Stresses

In addition to stresses associated with FML installation, the placement of overlying waste will impose stresses on the FML. Such stresses may result from disturbance of the FML during the waste disposal operations, elongation of the FML following potential foundation settlements, and rupture of the FML as a result of long term hydrostatic stresses.

Disturbance and/or displacement of the granular blanket during placement of the lowest 3 to 4 feet of waste material is a potential source of stress associated with waste disposal operations. Liner stress could result from movement of granular materials across the FML or from equipment coming in contact with the FML. Therefore, careful handling of this lowest layer of waste will help assure that the granular blanket stays in place, thereby providing maximum protection to the synthetic liner. To further minimize damage to the FML during filling operations, it is recommended that waste be placed as evenly as possible.

Elongation of the FML following settlement of foundation material has been evaluated. Calculations determining the extent of this elongation are shown in Appendix II, Figure 25. As shown in Figure 25, a conservative assumption that the maximum settlement occurs at the toe of the sideslopes (in reality, it would be expected to occur near the center of the cell), would result in tensile strains of approximately 3% in the FML. By contrast, the NSF standard specification for minimum elongation at yield of 80-mil HDPE is 10% and the NSF specification for minimum elongation at break is 500%, indicating that elongation due to settlement should not result in damage to the FML.

The integrity of the FML with respect to potential long-term hydrostatic stresses on the liner has also been considered. The liner would be subjected to maximum hydrostatic pressure in the case (considered unlikely) where the cell is fully saturated to the landfill cover and the leak detection system is fully operational. In this case, if the entire depth of waste is saturated, approximately 60 feet of hydraulic head would impart a hydrostatic pressure of 26 psi to the FML. Additionally, the total pressure due to saturated waste may reach a value approximately twice this level. These pressures are an order of magnitude less than the hydrostatic resistance reported by HDPE manufacturer's information for the 80-mil material.

10.5 Construction Requirements [299.9505 (1)(c)(i), 299.9621 (1)(d), 40 CFR 264303 (a)(1)]

Quality control and assurance requirements for installation of FML material used in the double liner system are detailed in the Construction Quality Assurance Plan document. These requirements reflect applicable Act 64 and RCRA regulations as well as manufacturer's specifications for handling and placement of FML.

11.0 LEACHATE COLLECTION SYSTEM [299.9505 (1)(2), 299.9619 (4), 40 CFR 264301 (A)(2)]

11.1 Elements [299.9619(4), 299.9505(1)(e)(i,ii)]

The proposed leachate collection and removal system is detailed on the engineering design drawings prepared by MCI. It will consist of a combination of granular drainage medium, synthetic filter/drainage pipes that empty into four leachate collection sumps. The granular drainage medium will consist of a 1-foot thick blanket of MDOT Class II sand placed over the base of the cell. The coefficient of permeability for the Class II sand will be a minimum of 1×10^{-2} cm/sec. Geotextile drainage net will be placed along the sideslopes of the cells in lieu of the granular blanket. Both the drainage net and Class II sand will be covered with a synthetic filter fabric.

Geotextiles in the leachate collection system serve several purposes. Filter fabric inhibits the entrance of fine particles into the pores of the leachate collection sand blanket. The drainage net along the sideslopes allows fluid transmission parallel to the plane of the netting and minimizes erosion and instability that could occur if sand functions as the sideslope drainage medium.

Perforated, 6-inch diameter, high density polyethylene (HDPE) pipe will be used in the leachate collection system. These pipes will be placed along the base of the cell within the Class II sand blanket and will have a standard dimension ratio of 7.3 and 1/4 inch diameter perforations. Each lineal foot of pipe will have 2 rows of perforations with individual perforations spaced at 4-inch intervals, i.e., 6 holes per lineal foot. Rows will be spaced 90° apart. Pipes within the Class II sand will be enveloped by MDOT Series 34 open graded aggregate (pea gravel). Filter fabric will be placed between the pea gravel and Class II sand to inhibit fines from migrating into the pea stone.

Leachate collection sumps will be constructed of HDPE materials. As shown on design plans, and in accordance with Michigan Act 64 Rule 299.9619 (4), design sumps will accommodate a leachate volume of not less than 4000 liters or the quantity expected to be generated during a 24-hour, 100-year storm frequency.

The following subsections evaluate various features of the leachate collection system. Evaluations are based on various manufacturers' published data for corresponding material properties used in the system. Presently, specific manufacturers of materials have not been selected. Therefore, it may be necessary to re-evaluate certain features of the leachate collection system if properties of materials selected for use are substantially different from the assumed properties on which these evaluations are based.

11.2 Pipe Spacing [299.9505(1)(e)(ii)(C-D), 299.9620 (4), 40 CFR 264.301(a)(2)]

An evaluation of maximum pipe spacing needed to limit hydrostatic head on the FML to less than or equal to 6 inches is presented as Figure 30 of Appendix II. This calculation suggests that the maximum length of flow in the drainage blanket to the nearest collection pipe should not exceed 110 feet. However, an evaluation of the transmissivity of the drainage blanket indicates pipes should not be spaced at greater than 48 feet. As shown on design plans, lateral collector pipes are therefore spaced at 45-foot intervals. This spacing will prevent the hydrostatic head from exceeding the 6-inch limit and allow the leachate collection system to meet transmissivity requirements discussed in Section 11.3.

Calculations in Figure 30 are based on a design infiltration rate of 2.3×10^{-6} cm/sec resulting from a water balance calculation presented in Figure 31 of Appendix II. This infiltration rate corresponds to a situation where an unvegetated intermediate cover overlies refuse.

11.3 Minimum Allowable Transmissivity [299.9505 (1)(e)(i)]

In addition to maximum allowable head buildup on the FML, minimum allowable transmissivity within the leachate collection system is another factor that governs the spacing of lateral drains. Calculations evaluating the transmissivity of the drainage net along cell sideslopes and Class II sand along the base are presented in Appendix II, Figures 32 through 35. Figure 32 suggests that a transmissivity of 0.34 cm^2/sec is needed to transmit design flows. However, as shown in Figure 32, the transmissivity of the Class II sand is approximately 0.15 cm^2/sec. Therefore, reduction of maximum pipe spacing is necessary to achieve the required transmissivity needed to accommodate design flows. As indicated in Figure 33, lateral drains spaced at approximately 48 feet along the base of Cell II will provide effective drainage. As shown on design plans, pipes will be spaced at 45-foot intervals.

As shown in Figure 22E, the minimum required transmissivity of the drainage net along the sideslopes is approximately 3.5×10^{-5} m^2/sec, approximately equal to the available transmissivity values reported by manufacturers of drainage net materials. Therefore, such material will effectively transmit design flows along the sideslopes. Figure 35 indicates that compression of drainage nets that would result in unacceptable reduction in transmissivity should not occur under the weight of refuse in Cell II.

11.4 Pipes Minimum Design Slope and Pipes Perforation Size [R299.9505 (1)(e)(ii)(C)]

Calculations performed to determine if the minimum design slopes of the leachate collection pipes are adequate to transmit infiltration entering the landfill are presented in Appendix II, Figure 41. These calculations are based on the design infiltration rate of 2.3×10^{-6} cm/sec presented in Figure 31. As shown in Figure 41, the minimum pipe slope required to transmit the maximum volume of leachate expected to be generated in the landfill is 0.13%, less than the proposed pipe slopes for the landfill.

Pipe perforation diameter has been evaluated to determine if perforations are large enough to accommodate anticipated design flows. Such an evaluation is made by calculating the entrance velocity of a flow entering the pipe. Entrance velocities exceeding 0.1 ft/sec suggest that perforation diameters are not large enough to accommodate design flow. As shown on Figure 42, Appendix II, entrance velocities do not exceed the limit of 0.1 ft/sec and the related pipe perforation diameters are adequate.

11.5 Filter Requirements [299.9505 (1)(e)(iii), 40 CFR 264.301 (a)(2)(ii)]

Filter criteria for pea gravel/collection pipe and Class II sand/drainage fabric interfaces have been evaluated. This evaluation is presented on Figure 43 of Appendix II. Filter criteria was used to examine the effectiveness of drainage fabric in transmitting leachate to the collector pipes as well as the potential for pea gravel to enter and clog the collector pipes. As shown in Figure 43, the relative size of pipe perforations with respect to the grain size of pea gravel should preclude pipe clogging. Calculations also show that the minimum permeability of filter fabric used in the drainage system should be at least 0.1 cm/sec.

11.6 Chemical Compatibility [299.9505 (1)(e)(ii)(E), 40 CFR 264.301 (a)(2)(i)(A)]

At this time, manufacturers of geosynthetics and pipes intended for use in the leachate collection system have not been selected. Following selection of a manufacturer, chemical property data for products used in the leachate collection system will be compiled and evaluated in order to assess chemical compatibility with the landfill environment. Any additional property data required to evaluate the integrity of components of the leachate collection system will also be obtained at this time.

12.0 CONSTRUCTION REQUIREMENTS FOR THE LEACHATE COLLECTION SYSTEM

General construction requirements for the leachate collection system are included in the Construction Quality Assurance Plan document.

13.0 FML QUALITY CONTROL AND ASSURANCE

The FML Installer (FMLI) shall install both the 80-mil thick high density HDPE secondary liner and the 80-mil thick HDPE primary liner. Both the primary and secondary liners shall be restrained at their upper edges at an anchor trench, with all trenches to be dug with a backhoe by the EWC. Seaming shall be accomplished using an extrusion-fusion type weld with a minimum sheet overlap of 4 inches at all seams. Extrusion-fusion type welding involves the use of a heat gun to heat the surface of the HDPE and extrude liquid HDPE that bonds individual panes.

The FMLI will also conduct its own quality control program which consists primarily of three parts: material testing at the point of manufacture, in-place seam integrity testing and destructive testing of seams. The CQA Officer shall appoint an independent testing engineer (ITE) to inspect the installation of the FML and to ensure that the FML quality control and quality assurance program outlined in this document is adhered to. Inspection performed by the ITE will supplement the FMLI's quality control program. The ITE will provide the CQA with information needed for the CQA to certify that it has been placed in accordance with cell engineering design plans.

13.1 FML Quality

13.1.1 Raw Material

a. The FML must be manufactured of first quality newly produced materials. The use of reclaimed polymers and other materials is not permitted. Recycling of materials containing reinforcing scrim is not permitted. Recycling of materials that do not contain scrim is permitted.

b. The following documentation relating the FML raw material quality must be provided by the FMLI.

- A statement identifying the origin of raw materials.

- A copy of the quality control certificates issued by the producer of raw materials.

- Reports on tests conducted to verify the quality of the raw materials.

13.1.2 <u>Manufactured Rolls and Blankets</u>

a. FML blankets or rolls must be designed and manufactured specifically for the purpose of fluid containment.

b. The FML must be free of holes, blisters, undispersed raw materials, and any sign of contamination of foreign matter.

c. The FML used as the secondary liner must be a minimum of 80-mils thick.

d. The FML used as the primary liner must be a minimum of 80-mils thick.

e. The 80-mil FMLs will have the material properties listed in Standard 54 of the National Sanitation Foundation (NSF) for HDPE material.

f. The following documentation relating to quality of manufactured FML rolls and blankets will be provided by the FML manufacturer:

- <u>Material Property Sheets</u> - These sheets will pertain to the FML to be used for the project and will contain results of tests to verify compliance with the minimum acceptable standard properties specified by Standard 54 of the NSF. The sheet must also provide any minimum properties guaranteed by the FML manufacturer and indicate the test method used.

- <u>Quality Control Certifications</u> - Certificates will pertain to rolls or blankets of material delivered to the site and will be signed by a responsible party employed by the FML manufacturer such as production manager.

- Each roll or blanket will be identified by a unique manufacturing number.

13.1.3 <u>Factory Seaming</u> - If factory seaming is performed, the FMLI will provide documentation of seaming conditions and the test results of factory seams.

13.2 Packaging, Storage and Handling of FML Prior to Installation

- FML rolls or blankets must be packed and labeled prior to shipment to the site. The label must indicate the FML manufacturer, type of FML, thickness, and roll or blanket number.
- When transported to the site, FML rolls or blankets must be handled by appropriate means so that no damage is caused. Wooden cases must be strong enough to withstand impacts and rough handling without breaking or splintering.

- The FML must be protected from direct sunlight and heat to prevent degradation of the FML material and adhesion between individual whorls of a roll or layers of a blanket. Adequate measure must be taken to keep FML materials away from possible deteriorating sources.

- On site, the FML will be stored in an area away from heavy traffic.

- Appropriate handling equipment must be used when moving the rolled or folded FML from one place to another.

13.3 Preparation of Subgrade

- The subgrade for the secondary FML will consist of compacted clay fill, where necessary, along the base and sideslopes of Cell II shaped according to the subgrade plans.

- The subgrade for the primary FML will consist of the 5-foot compacted clay primary liner overlying the secondary FML and the leak detection system.

- Subgrades below the FML will be graded to eliminate protruding stones and deleterious materials. Deviation in design grade elevations of more than 0.2 feet are unacceptable.

- The upper three inches of the layer must not contain protruding stones larger than 2 inches in diameter. Large stones will be removed by hand at the time of fill placement and preparation of cut slopes where applicable.

- A smooth steel drum, pneumatic roller or other approved piece of equipment will be used to free subgrade surfaces of irregularities, loose earth and abrupt changes in grade.

- No FML may be placed in ponded precipitation or in any area which has become softened by precipitation.

- The CQA Officer will make provisions for material, personnel and equipment needed to prepare and maintain an acceptable subgrade surface.

13.4 Installation of FML

13.4.1 General Installation

a. FML rolls or blankets may be cut into panels, a unit area of membrane which is to be seamed. Individual panels will be designated a panel number. Instruction on the boxes or wrapping containing the FML materials must be followed to assure the panels are unrolled or unfolded in the proper direction for seaming. Care must be exercised to not damage the FML during this operation. All workers must wear shoes which will not damage the FML.

b. FML panels will be placed according to FML layout drawings prepared by the FMLI prior to placement. The drawings must indicate the panel configuration and locations of seams. Field seams should be differentiated from factory seams (if any). In general, seams should be oriented parallel to line of maximum slope. In corners and odd shaped geometric locations, the total length of field seams should be minimized. No horizontal seams should be placed at the toe and should be a minimum of 1.5 m (5 feet) away from the toe of the slopes.

c. Pulling FML panels will be minimized to reduce permanent tension.

d. The following precautions will be taken to minimize the risk of damage by wind during panel placement.

- No more than one panel should be unrolled prior to seaming (unless otherwise authorized by the installer).

- FML panels will be secured to prevent uplift by the wind during placement. Sand bags, tires or any other means which will not damage the FML will be used to secure it. Along the edges, loading must be continuous to avoid possible wind flow under the panels.

e. Any panel which becomes seriously damaged (torn or twisted permanently) must be replaced. Less serious damage must be repaired according to Section XII-F.

f. FML placement must not proceed at an ambient temperature below 1°C (34°F) or above 35°C (95°F) unless approved by the FMLI and CQA Officer.

g. FML placement must not occur during precipitation events.

13.4.2 <u>Installation Around Appurtenances</u>

a. The FML must be installed around the leachate collection manhole and an FML sleeve must initially be installed around the HDPE manhole riser. After the FML has been placed and seamed, the final field seam connection between the sleeve or shield and the FML liner must be completed. A sufficient initial overlap of the sleeve must be maintained so that shifts in locations of the FML can be accommodated.

b. All clamps, clips, bolts, nuts or other fasteners used to secure the FML around each appurtenance must be made of stainless steel material.

13.5 Field Seaming and Testing

13.5.1 <u>General Requirements</u>

a. Panels shall be overlapped a minimum of 4 inches (100 mm).

b. Prior to seaming, the seam area must be cleaned of dust, dirt, debris of any kind, and foreign materials.

c. Seaming products shall be formulated in accordance with the FML manufacturer's specifications.

d. Seaming will be performed under favorable weather conditions.

e. Seaming on horizontal surfaces must commence at the center of a panel side and proceed to either side (if possible) in an effort to reduce wrinkle and subsequent fishmouths at the seam interface. Seaming shall extend to the outside edge of panels.

f. If the supporting soil is soft, a firm substrate must be provided by using a board or similar hard surface directly under the seam overlap to effect proper rolling pressure.

g. No loose flap of FML will be permitted on the upper surface of the completed installation.

h. Fishmouths or wrinkles at the seam overlaps must be cut along the ridge of the wrinkle back into the panel so as to effect a flap overlap. The cut fishmouths or wrinkles must be seamed and then patched with an oval or round patch of the same general FML extending a minimum of 6 inches (150 mm) beyond the cut in all directions. The patch must be bonded over its entire perimeter.

13.5.2 <u>Start-up Field Test Seams</u>

a. Test seams must be performed to verify that seaming conditions are adequate. Test seams shall be conducted at least two times each day (at the beginning of the morning and the beginning of the afternoon) for each seaming method used that day. Test seams will be performed under the same conditions that panel seams are performed. Per ASTM D4437, the test seams must be at least 10 feet (3 m) long.

b. Specimens must be cut from the test seam. A 2-inch wide strip will be cut perpendicular to the seams. Specimens will be tested for shear strength by manually pulling each end of the strip. Test seams will be tested for peel strength by manually pulling the flap on the underside of the strip, away from the strip. If the test seam fails, an additional test seam shall immediately be conducted. If the additional test seam fails, the seaming equipment or product must be rejected and not used for production seaming until the deficiencies are corrected and a successful test seam is produced.

c. A sample from each test seam must be retained and labeled with the date, ambient temperature, number of seaming unit, seamer, and pass or fail description. One half of the sample must be given to the FMLI and the other shall be retained by Ford.

13.5.3 <u>Non-destructive Field Seam Testing</u>

a. All field seams must be non-destructively tested over their length. Each seam must be numbered or otherwise designated. The installer shall document the results of the non-destructive testing.

b. Testing must be done as the seaming progresses, not at the completion of all field seaming. All defected found during testing must be numbered and marked immediately after detection. All defects found must be repaired, retested and remarked to indicate completion of the repair acceptability.

c. All seams shall be fully tested by vacuum test methods, except as noted in Item 4 below.

d. All seams in special locations must be non-destructively tested if the seams are accessible to testing equipment. If any seam cannot be non-destructively tested, that seam must be observed by the CQA or his representative for uniformity and completeness and shall be so documented by the CQA.

13.5.4 <u>Destructive Laboratory Seam Testing</u> - Destructive seam testing shall be performed at a minimum of one destructive test per 350 feet of field seam length at locations to be determined by the CQA Officer. The samples shall be 16 inches wide by 24 inches long. One-half of the sample will be retained by the CQA Officer or his representative and one-half will be retained by the FMLI. The FMLI will perform five laboratory tests for shear and peel strength on specimens cut from the seam sample. Four of the five replicate test results must pass the material specification requirements of the NSF Standard 54. Results of destructive testing shall be supplied to the CQA or his representative. In the event of destructive test failures, the FMLI shall determine the length of seam failure to the satisfaction of the CQA Officer or his representative. The area of failure must be reseamed or cap stripped. Test methods shall be: Shear Strength Test ASTM D816-Method B; Peel Strength Test ASTM D413-Method H, or ASTM D816-Method C.

13.6 Repair of Defects

- All seams and non-seam areas of the FML must be inspected for identification of defects, holes, blisters, undispersed raw materials and any sign of contamination by foreign matter.

- The surface of the FML shall be clean prior to use. Sweeping and/or washing of the FML surface is required if the amount of surface dust or mud inhibits inspection.

- Repairs will be made in non-seam areas having defects, holes, blisters, undispersed raw material or any sign of contamination and on seams that have failed non-destructive testing.

- Defective seams must be repaired by reseaming or applying a cap-strip. Tears or pinholes must be repaired by patching. Blisters, larger holes, undispersed raw materials, and contamination by foreign matter shall be repair by patches. Each patch must be numbered. patches must be round or oval in shape, made of the same generic FML and extend a maximum of 6 inches (150 mm) beyond the edge of defects.

- Cap-strips must be at least 3 inches (75 mm) wide and must be centered over the completed seam edge. Cap-strips must be of the same generic FML material as the liner.

- The thickness of cap-strip material used on the secondary FML must be at least 60 mils. The thickness of cap-strip materials used on the primary FML must be at least 80 mils.

- Each repair must be non-destructively tested using the methods described in Section 13.5. Tests which pass the non-destructive tests are taken as an indication of an adequate repair. Failed tests must be reseamed and retested until a passing test results. The results of all non-destructive testing performed on cap-strips must be documented.

13.7 Qualifications and Responsibilities of the FMLI

- The FMLI must be trained and qualified to install 80-mile HDPE synthetic liners.

- To demonstrate the necessary training and qualifications, the FMLI must provide to Ford the following information about at least three previous projects: name and purpose of the project; location; date; names of owner, designer and manufacturer; leader of the installer's crew; type of FML; thickness of FML; surface area; type of seaming; duration of installation; and available written information on the performance of the project.

- FMLI personnel involved in field seaming operations must be qualified by experience or by successfully passing seaming tests.

- At least one seamer must have experience seaming at least 100,000 square meters (1.07 million square feet) of an FML of the same generic type as the FML used for the project using the same type of seaming method. This master seamer must provide direct supervision over apprentice seamers.

- Apprentice seamers must be qualified by attending training sessions taught by the master seamer and performing at least two successful seaming tests under similar weather conditions using the seaming method used for production seaming.

- The FMLI must provide to Ford documentation indicating that the personnel involved in field seaming operations have experience and qualifications as outlined in Items 3 to 5 above.

- The FMLI will be responsible for receipt, inspection and handling of FML materials as well as testing and repairing the FML when necessary.

- The FMLI will provide to the CQA all documents relating to the quality of FML raw materials as well as manufactured rolls or blankets. The FMLI will provide the CQA with the following information:

  a. A statement identifying the origin of the raw materials.

  b. Quality control certificates issued by the producer of the raw materials.

c. Reports on tests conducted to verify the quality of raw materials.

- Upon arrival of the FML at the site, the FMLI will inspection all material for defects in manufacturing.

- The FMLI must ensure that the following information is provided for rolls or blankets of FML material arriving on site:

  a. material property sheets

  b. quality control certificates

- The FMLI must ensure that each FML roll or blanket arriving on site is labeled with the following information:

  a. FML manufacturer

  b. type of FML

  c. thickness of FML

  d. roll or blanket number

- The FMLI must provide the CQA and the ITE a layout drawing of the proposed FML placement pattern and seams prior to FML placement.

- The FMLI must inspect each FML panel for defects following placement and prior to seaming.

- The FMLI must verify that the weather condition are acceptable for seaming. Ambient temperature and liner temperature will be recorded by the FMLI hourly during liner installation and field seaming.

- The FMLI must provide suitable seaming equipment and products needed for seaming operations.

- The FMLI will perform FML placement, seaming, test seaming, non-destructive testing and repairing according to procedures as outlined in Subsections D through F of this section.

- The FMLI will record the following information for all non-destructive seam testing that is performed:

    a. location of non-destructive testing

b. date

c. test unit

d. name of tester

e. result of testing

- The FMLI will record all information included under above item for non-destructive seam testing performed on all repairs.

- The FMLI must retain a sample from each test seam and label it with the date, ambient air temperature, number of seaming unit, name of seamer, and result of test.

- The FMLI must provide to Ford daily reports including the following information:

a. total amount and location of FML placed

b. total amount of seams completed and seaming units used

c. changes in layout drawings

d. results of test seams

e. location and results of non-destructive testing

f. locations and results of repairs

g. location of and results of destructive seam testing, if performed

h. acceptance of subgrade

13.8 Responsibilities of the ITE

- The ITE or his representative will observe and document all field seaming operations including weather conditions, FML cleaning, overlaps, rate of seaming, names of seamers and seaming units used. He will also be on site to observe and document other phases of FML installation.

- The ITE or his representative will also observe and document the phases of the FML installation that will include but not be limited to:

  a. Acceptability of subgrade preparation prior to the installation of the FML.

b. Observations of test seams and non-destructive seam testing.

c. Observations of repairs and testing including locations, name of repairer and seaming equipment of product used.

d. Observations of seams around appurtenances and connections to appurtenance.

e. The above observations will be communicated to the CQA Officer through the submittal of a daily field report.

14.0 CONSTRUCTABILITY OF THE CELL AND DOUBLE LINER SYSTEM

14.1 Suitability of Materials

Results of laboratory testing performed on both the native on-site clay and the I-696 clay indicate that these materials meet the compaction, moisture, permeability and strength requirements for use in the double liner system and in the preparation of Cell II. Material used in the preparation of Cell II will be placed in the compacted clay subbase, U-shaped stabilization berms, perimeter berms and sideslope fill. Material used in the construction of the double liner system will comprise the 5-foot compacted clay layer.

14.1.1 ON-SITE CLAY - Laboratory test data for native on-site clay presented in Tables 1, 2 and 4 suggest that this material may be used in either construction of the 5-foot compacted clay liner or in the preparation of Cell II. As shown on Table 4, the permeability of this material exceeds 1 x 10^{-7} cm/sec in samples prepared at 90% compaction and in samples prepared at 95% compaction and moisture contents below optimum. As indicated in Section 9.0 of this document, the permeability of material used in the double liner system cannot exceed 1 x 10^{-7} cm/sec. Therefore, it is recommended that any native on-site clay used in the construction of the 5-foot compacted clay liner (along side slopes or inside stabilization berms) should be compacted to 95% of the maximum dry density and at moisture contents above optimum. On the other hand, if this material is used in the construction of the compacted clay subbase, U-shaped stabilization berms, side berms or in the placement of sideslope fill, compaction to 90% of maximum dry density and at moisture contents ranging from 2% below to 5% above optimum will be adequate.

It should be noted that native on-site clay has been used in the construction of the Cell I Final Cover. During placement of the cover, NTH measured in-place dry density and moisture content of the compacted clay using a nuclear densometer (ASTM D1557). Field Density Test Report dated 6/18/87 and submitted by NTH indicated that compaction of 95% and moisture contents above optimum is achievable in the field using D-6 and D-8 equipment.

14.1.2 <u>I-696 CLAY</u> - Laboratory test data for the I-696 clay presented in Tables 1, 3 and 4 suggest that this material may be used in either construction of the 5-foot compacted clay liner or in the preparation of Cell II. As shown on Table 3, shear strengths of this materials are less than 2500 psf in samples prepared at moisture contents of 5% above optimum. As indicated in Section 9.0 of this document, a minimum shear strength of 2500 psf is required of material placed in the 5-foot compacted clay layer on cell sideslopes, on fill placed on all sideslopes, in the U-shaped stabilization berm and in the side perimeter berm. Therefore, it is recommended that I-696 clay used in the areas be placed at 90% compaction and moisture contents ranging between 2% below to 3% above optimum. I-696 clay placed in all other portions may be placed at 90% compaction and moisture contents ranging from 2% below to 5% above optimum.

It should be noted that I-696 clay has been used in the construction of the Cell I Final Cover. During placement of the cover, NTH measures in-place dry density and moisture content of the compacted clay using a nuclear densometer (ASTM 1557). Field density test reports prepared by NTH during the spring of 1987 indicate that compaction and moisture content requirements are achievable in the field using D-6 and D-8 equipment.

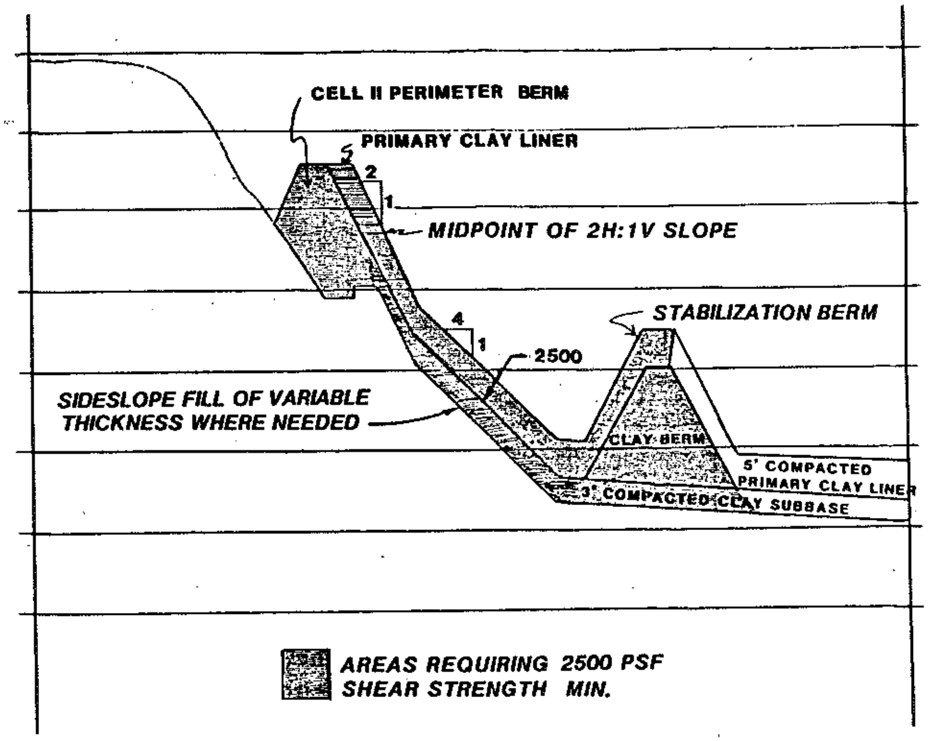
14.2 Suitability of Construction Methods

It is anticipated that construction activities will proceed in a manner that will ensure the integrity of the Cell II double liner system. As indicated in the Construction Quality Assurance Plan dated June 24, 1988, field testing will be performed at regular intervals to ensure that specified moisture/density requirements are met. As stated in the above section, field moisture/density testing performed on native on-site clay and I-696 clay placed in the Cell I cover demonstrates that moisture/density requirements can be met with existing construction equipment.

The Construction Quality Assurance Plan also contains provisions for regular determinations of the permeability and shear strength of the compacted clay material. Also, provisions are included in the Plan for the removal of deleterious material prior to placement of the clay to ensure that field moisture/density, permeability and shear strength of the compacted clay are not adversely affected.

It is anticipated that construction equipment used in the construction of the double liner system will not damage components of the double liner system. Throughout this document, provisions have been made to protect various liner components during construction. Low ground pressure equipment is specified to prevent damage to underlying geosynthetics during the placement of the 5-foot compacted clay liner and the granular blanket of the leachate collection system. In addition, quality control procedures specify that the first lift of material placed have a loose thickness of 18-inches to protect underlying geosynthetics. Loose thicknesses of subsequent lifts are specified at 9 inches to ensure uniform compaction throughout the liner and adequate bonding between lifts. Finally, as indicated in Section 8.3 of this document, the results of a previous test fill performed at another site using a similar liner design demonstrate that earth moving equipment and proposed construction methods are not expected to damage underlying geosynthetics or cause excessive stresses during construction of the double liner system.

PERMEABILITY (K) & SHEAR STRENGTH (S) SPECIFICATIONS FOR CONSTRUCTION PHASES	MOISTURE / DENSITY REQUIREMENTS	
	NATIVE ONSITE CLAY	I-696 CLAY
1. Cell II Subbase Fill within Stabilization Berm K = 1 X 10^{-6} cm/s S = 500 psf	90% -2% → + 5%	90% -2% → + 5%
2. Cell II Stabilization Berms K = 1 X 10^{-6} cm/s S = 2500 psf	90% -2% → + 5%	90% -2% → + 3%
3. Cell II Sideslope Fill K = 1 X 10^{-6} cm/s S = 2500 psf	90% -2% → + 5%	90% -2% → + 3%
4. Cell II Perimeter Berms K = 1 X 10^{-6} cm/s S = 2500 psf	90% -2% → + 5%	90% -2% → + 3%
5. Five-Foot Compacted Clay Liner along Cell Sideslopes K = 1 X 10^{-7} cm/s S = 2500 psf	95% opt. → + 5%	90% -2% → + 3%
6. Five-Foot Compacted Clay Liner inside Stabilization Berms K = 1 X 10^{-7} cm/s S = 500 psf	95% opt. → + 5%	90% -2% → + 5%



CELL II LINER SYSTEM CONSTRUCTION REQUIREMENTS
ALLEN PARK CLAY MINE
FORD MOTOR COMPANY
DEARBORN, MICHIGAN

NTH NEYER, TISEO & HINDO, LTD.
CONSULTING ENGINEERS AND GEOLOGISTS
38955 HILLS TECH DRIVE • FARMINGTON HILLS, MI 48018

PROJECT NO.: 86347OW	DRAWN BY: FK	DATE: 3-30-88
SCALE: - NO SCALE	CHECKED BY: CJM	SHEET OF

PLATE 1

TABLE 1

CHARACTERISTICS OF CLAY SOURCES

SOURCE SAMPLE DESIGNATION	MODIFIED PROCTOR RESULTS		CHARACTERISTICS		
	MAX. DRY DENSITY (lb/ft^3)	OPTIMUM MOISTURE CONTENT (%)	% PASSING 200 SIEVE	LIQUID LIMIT	PLASTICITY INDEX
ON-SITE:					
Bag 1 6/12/86	125.7	12.0	82.8	31	14
Bag 2 6/12/86	123.2	12.8	99.2	31	12
ST-1 6/18/86	---	--	98.8	34	19
ST-2 6/18/86	---	--	86.5	37	18
ST-3 6/18/86	---	--	99.4	34	19
Bag 1B 6/86	129.7	10.4	71.0	21	8
Bag 2B 6/86	129.1	9.2	88.0	31	14
Bag 1C 4/10/87	121.5	11.1	99.2	31	12
Bag 2C 4/10/87	113.6	14.9	99.6	31	16

TABLE 1 (cont.)

CHARACTERISTICS OF CLAY SOURCES

SOURCE SAMPLE DESIGNATION	MODIFIED PROCTOR RESULTS		CHARACTERISTICS		
	MAX. DRY DENSITY (lb/ft^3)	OPTIMUM MOISTURE CONTENT (%)	% PASSING 200 SIEVE	LIQUID LIMIT	PLASTICITY INDEX
I-696:					
Bag 1C 4/10/87	132.6	10.4	72.3	29	14
Bag 2C 4/10/87	131.6	10.3	65.4	28	13
Bag 5 7/31/86	131.4	8.0	64.4	25	9
Bag 8 8/13/86	130.6	10.0	70.2	27	12
Bag 10 9/04/86	120.2	13.6	--	--	--
Bag 13 11/3/86	---	--	68.5	26	11

TABLE 2
RESULTS OF LABORATORY STRENGTH TESTING - NATIVE ON-SITE CLAY

SAMPLE NUMBER	DRY DENSITY (lb/ft^3)	% OF MODIFIED PROCTOR MAXIMUM	MOISTURE CONTENT (%)	COMPARISON TO OPTIMUM MOISTURE CONTENT (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	UNDRAINED SHEAR STRENGTH (psf)	STRAIN FAILURE (%)
1A	119.7	93	15.1	+5.9	7400	3700	19.0
2A	119.2	92	15.3	+6.1	9354	4677	17.6
3A	119.9	93	14.9	+5.7	8544	4272	19.4
4A	120.2	93	14.9	+5.7	8738	4369	19.4
5A	102.5	93	14.6	+5.4	8212	4106	19.4
6A	120.2	93	14.7	+5.5	8074	4037	19.4
7A	120.4	93	14.5	+5.3	7716	3858	19.4
8A	120.2	93	14.7	+5.5	8324	4162	19.4
9A	120.7	93	14.3	+5.1	8654	4327	19.4
10A	112.1	87	12.5	+3.3	8042	4021	3.5
11A	112.8	87	12.1	+2.9	7214	3607	4.4
12A	112.7	87	11.9	+2.7	7116	3558	3.5
13A	115.4	89	12.0	+2.8	8348	4174	2.6
14A	115.4	89	12.0	+2.8	9510	4755	4.4
15A	115.7	90	11.9	+2.7	8198	4099	4.4
16A	119.6	93	11.7	+2.5	12672	6336	6.2
17A	119.7	93	11.6	+2.4	10884	5442	5.3
18A	119.3	92	11.9	+2.7	10884	5442	5.3
19A	114.0	88	17.4	+8.2	2264	1132	19.4
20A	113.7	88	17.7	+8.5	1714	857	19.4
21A	114.1	88	17.3	+8.1	2046	1023	19.4
22A	110.7	86	18.7	+9.5	1106	553	19.4
23A	110.9	86	18.6	+9.4	1162	581	19.4
24A	110.7	86	18.7	+9.5	1162	581	19.4

TABLE 3
RESULTS OF LABORATORY STRENGTH TESTING - I-696 CLAY

SAMPLE NUMBER	DRY DENSITY (lb/ft^3)	% OF MODIFIED PROCTOR MAXIMUM	MOISTURE CONTENT (%)	COMPARISON TO OPTIMUM MOISTURE CONTENT (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	UNDRAINED SHEAR STRENGTH (psf)	STRAIN FAILURE (%)
1A	115.2	86.9	8.8	-1.6	5940	2970	1.8
2A	115.9	87.4	8.5	-1.9	6710	3355	1.8
3A	115.9	87.4	8.4	-2.0	7940	3970	1.8
4A	122.3	92.2	8.8	-1.6	8910	4455	1.8
5A	122.5	92.4	9.0	-1.4	9680	4840	1.8
6A	122.4	92.3	8.8	-1.6	10610	5305	1.8
7A	125.3	94.5	10.0	-0.4	11430	5715	4.5
8A	125.5	94.7	10.0	-0.4	12520	6260	4.5
9A	125.8	94.9	9.9	-0.5	18260	9130	5.4
1B	123.2	92.3	10.2	-0.2	10060	5030	3.6
2B	123.2	93.0	10.4	0.0	9730	4865	3.6
3B	123.1	92.8	10.4	0.0	11850	5925	4.5
4B	125.8	94.9	10.4	0.0	16720	8360	4.5
5B	126.0	95.0	10.4	0.0	16024	8012	4.5
6B	126.0	95.0	10.3	-0.1	15240	7620	4.5
7B	126.7	95.5	10.4	0.0	14400	7200	5.4
8B	126.8	95.6	10.5	+0.1	15160	7580	4.5
9B	127.0	95.7	10.5	+0.1	14690	7345	4.5
1C	115.6	87.8	11.1	+0.8	9920	4960	2.7
2C	116.6	88.6	10.7	+0.4	10510	5255	1.8
3C	117.8	89.5	9.4	-0.9	10260	5130	2.2
4C	124.5	94.6	13.0	+2.6	10190	5095	19.6
5C	124.7	94.8	12.9	+2.5	11180	5590	19.6
6C	125.2	95.1	12.5	+2.1	10300	5150	19.6
7C	115.4	87.7	15.6	+5.2	2050	1025	19.6
8C	115.6	87.8	15.9	+5.5	2320	1160	19.6
9C	116.0	88.2	15.6	+5.2	2570	1285	19.6
1D	118.0	89.0	15.6	+5.2	1640	820	15.6
2D	117.9	88.9	15.6	+5.2	2050	1025	19.6

TABLE 4

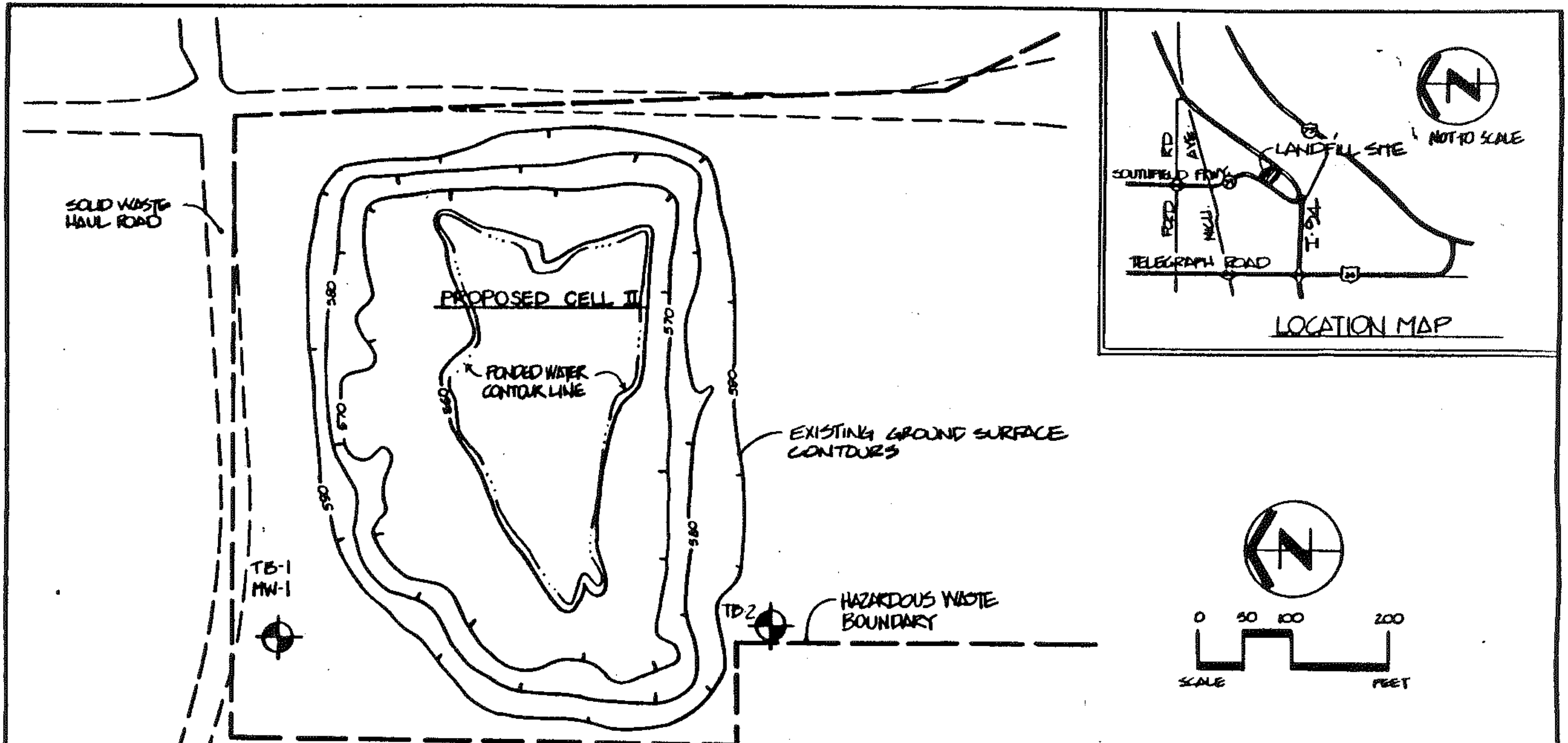
SUMMARY OF PERMEABILITY TEST RESULTS - CLAY SOILS

SAMPLE SOURCE DESIGNATION & DATE COLLECTED	TESTING DEVICE	% COMPACTION	MOISTURE CONTENT (%)	COMPARISON TO OPTIMUM MOISTURE CONTENT (%)	PERMEABILITY COEFFICIENT $x10^{-7}$ cm/sec
ON-SITE:					
Bag #1	(1)	89.8	10.4	-1.6	0.98
6/12/86		90.4	11.9	-0.1	0.84
		90.9	16.3	+4.3	0.98
		94.8	10.2	-1.8	0.55
		95.3	11.5	-0.5	0.38
		95.3	16.4	+4.4	0.08
Bag #2	(1)	90.1	10.5	-2.3	1.20
6/12/86		90.4	12.6	-0.2	1.30
		90.1	17.0	+4.2	0.07
		94.8	10.1	-2.7	0.66
		95.0	12.7	-0.1	0.94
		95.0	17.0	+4.2	0.05
Bag #1C	(1)	90.6	16.0	+4.9	3.60
4/10/87		95.4	10.6	-0.5	1.80
		94.0	16.4	5.3	0.10
I-696:					
Bag #1C	(1)	90.1	9.8	-0.6	2.30*
4/10/87		90.7	9.9	-0.5	0.77
		89.6	16.2	+5.8	0.12
		90.2	9.0	-1.4	0.47
Bag #5		90.0	6.0	-2.0	0.60
7/31/86		90.0	8.0	0.0	0.82
		90.0	13.0	+5.0	0.09
		95.0	6.0	-2.0	0.42
		95.0	8.0	0.0	0.47
		95.0	13.0	+5.0	0.53
Bag #28		90.0	9.3	-0.5	0.61
5/2/87		90.4	7.6	-2.2	0.48
		90.0	14.2	+4.4	0.07
Bag #28		90.1	8.9	-2.2	0.46
		90.2	10.6	-0.5	0.21
		92.1	15.0	3.9	0.08

(*) Test rerun and gave a value of 0.77

APPENDIX I

SUPPLEMENTAL SUBSOIL INVESTIGATION DATA



LEGEND:

TB — TEST BORINGS BY AMERICAN DRILLING CO. UNDER THE FULL-TIME SUPERVISION OF NEYER, TISEO & HINDO LTD. ON DEC. 20, 21 AND 26, 1984. TEMPORARY GROUNDWATER OBSERVATION WELL INSTALLED IN TB-1.

NOTES:

1) TEST BORING LOCATIONS SHOWN ARE APPROXIMATE

TEST BORING LOCATION PLAN

ALLEN PARK CLAY MINE LANDFILL
FORD MOTOR COMPANY
ALLEN PARK, MICHIGAN

NTH NEYER, TISEO & HINDO, LTD.
CONSULTING ENGINEERS
30999 TEN MILE RD. · FARMINGTON HILLS, MI 48024

PROJECT NO.: 84185 ON	DRAWN BY: MWW	DATE: 2-14-85
SCALE: AS SHOWN	CHECKED BY: WRB	SHEET 1 OF 1

PLATE 1

NEYER, TISEO & HINDO, LTD.

GENERAL NOTES

TERMINOLOGY

Unless otherwise noted, all terms utilized herein refer to the Standard Definitions presented in ASTM D 653.

PARTICLE SIZES

Boulders		-	Greater than 12 inches (305mm)
Cobbles		-	3 inches (76.2mm) to 12 inches (305mm)
Gravel	Coarse	-	3/4 inches (19.05mm) to 3 inches (76.2mm)
	Fine	-	No. 4 - 3/16 inches (4.75mm) to 3/4 inches (19.05mm)
Sand	Coarse	-	No. 10 (2.00mm) to No. 4 (4.75mm)
	Medium	-	No. 40 (0.425mm) to No. 10 (2.00mm)
	Fine	-	No. 200 (0.074mm) to No. 40 (0.425mm)
Silt		-	0.005mm to 0.074mm
Clay		-	Less than 0.005mm

COHESIONLESS SOILS

Classification

The major soil constituent is the principal noun, i.e. sand, silt, gravel. The second major soil constituent and other minor constituents are reported as follows:

Second Major Constituent (percent by weight)	Minor Constituents (percent by weight)
Trace - 1 to 12%	Trace - 1 to 12%
Adjective - 12 to 35% (clayey, silty, etc.)	Little - 12 to 23%
And - Over 35%	Some - 23 to 33%

Density Classification	Relative Density %	Approximate Range of (N)
Very Loose	0-15	0-4
Loose	16-35	5-10
Medium Compact	36-65	11-30
Compact	66-85	31-50
Very Compact	86-100	Over 50

Relative Density of Cohesionless Soils is based upon the evaluation of the Standard Penetration Resistance (N), modified as required for depth effects, sampling effects, etc.

COHESIVE SOILS

If clay content is sufficient so that clay dominates soil properties, clay becomes the principal noun with the other major soil constituent as modifier; i.e., silty clay. Other minor soil constituents may be included in accordance with the classification breakdown for cohensionless soils; i.e., silty clay, trace of sand, little gravel.

Consistency	Unconfined Compressive Strength (psf)	Appromixate Range of (N)
Very Soft	Below 500	0-2
Soft	500-1000	3-4
Medium	1000-2000	5-8
Stiff	2000-4000	9-15
Very Stiff	4000-8000	16-30
Hard	8000-16000	31-50
Very Hard	Over 16000	Over 50

Consistency of cohesive soils is based upon an evaluation of the observed resistance to deformation under load and not upon the Standard Penetration Resistance (N).

SAMPLE DESIGNATIONS

- AS - Auger Sample - Directly from auger flight.
- BS - Miscellaneous Samples - Bottle or Bag.
- S - Split Spoon Sample with Liner Insert - ASTM D 1586
- LS - Liner Sample S with liner insert 3 inches in length.
- ST - Shelby Tube Sample - 3 inch diameter unless otherwise noted.
- PS - Piston Sample - 3 inch diameter unless otherwise noted.
- RC - Rock Core - NX core unless otherwise noted.

STANDARD PENETRATION TEST (ASTM D 1586) - A 2.0" outside-diameter, 1-3/8" inside-diameter split barrel sampler is driven into undisturbed soil by means of a 140-pound weight falling freely through a vertical distance of 30 inches. The sampler is normally driven three successive 6-inch increments. The total number of blows required for the final 12 inches of penetration is the Standard Penetration Resistance (N).

LOG OF SUBSURFACE PROFILE

CLASSIFICATIONS BY: NEYER, TISEO & HINDO, LTD.

GROUND SURFACE ELEVATION: 595.6

ELEVATION-FEET

590 – 580: FILL: Black SILTY SAND AND GRAVEL with Wood, Wire, Brick, Concrete and Black Foundry Sand. (13.0)

580 – 500: Soft to Medium Gray SILTY CLAY with Trace of Sand. (95.0)

500: Very Compact Gray SAND AND GRAVEL. (100.0)

NOTES:
1. Borings advanced using 4-inch diameter solid-stem augers to 10 feet, and 3-7/8 inch diameter tricone roller bit with recirculating drilling fluid to bottom of hole. 4-inch diameter casing was driven to 12.5 feet.
2. Artesian water pressure was observed after penetrating the hardpan layer. No piezometric level was ob-
3. 2-inch diameter well installed. See Log .r Monitoring Well No. MW-1.

SOIL SAMPLE DATA

SAMPLE NUMBER	ELEV. (FEET)	NATURAL MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	PENETRATION* RESISTANCE
LS-1	590.6	12.4	101.4	6-5-5
LS-2	585.6	-	-	7-6-6
LS-3	580.6	-	-	2-3-4
PS-1	578.1	23.9	105.9	PUSHED
LS-4	575.6	-	-	1-1-2
LS-5	570.6	36.6	84.7	1-1-1
PS-2	563.1	17.9	116.4	PUSHED
VS-1	560.6	-	-	
LS-6	555.6	-	-	1-2-3
LS-7	550.6	21.1	107.9	1-2-3
LS-8	545.6	-	-	2-2-3
VS-2	540.6	-	-	
PS-3	535.1	21.7	110.3	PUSHED
S-1	530.6	-	-	2-3-4
VS-3	525.6	-	-	
LS-9	520.6	25.9	99.7	3-3-3
LS-10	515.6	-	-	2-3-4
LS-11	510.6	21.7	106.2	5-8-11
LS-12	505.6	-	-	4-5-6
LS-13	500.6	31.6	88.5	4-4-7
LS-14	495.6	NO RECOVERY		19-32-70/5"

PENETRATION RESISTANCE scale: 0 10 20 30 40

TOTAL DEPTH: 100.0'
BORING STARTED: 12/19/84
BORING COMPLETED: 12/26/84
INSPECTOR: L. Kendall/D. Vensel
DRILLER: J. Blank
CONTRACTOR: American Drilling Co.

WATER LEVEL IN HOLE AT INDICATED NUMBER OF HOURS AFTER COMPLETION OF BORING WITH 0 FEET OF CASING IN PLACE.

*PENETRATION RESISTANCE: NUMBER OF BLOWS REQUIRED TO DRIVE 2 INCH O.D. SOIL SAMPLER 12 INCHES, USING 140 POUND WEIGHT WITH 30 INCH FREE FALL.

NEYER, TISEO & HINDO, LTD.
CONSULTING ENGINEERS

LOG OF TEST BORING NUMBER TB-1

ALLEN PARK CLAY MINE LANDFILL
FORD MOTOR COMPANY
ALLEN PARK, MICHIGAN

APPROVED BY:	DATE: 1/11/85
PROJECT NO. 84185 OW	FIGURE NO. 1

LOG OF SUBSURFACE PROFILE

CLASSIFICATIONS BY: NEYER, TISEO & HINDO, LTD.

GROUND SURFACE ELEVATION: 591.9

ELEVATION-FEET

Depth	Description
2.5	FILL: Compact Dark Brown SILTY SAND with Slag.
9.0	FILL: Loose to Medium Compact Gray SILTY FINE SAND with Little Clay.
76.5	Soft to Medium Gray SILTY CLAY with Trace of Sand.
80.0	Very Hard Gray CLAYEY SILT with Some Fine Sand and Gravel. (Hardpan)

SOIL SAMPLE DATA

SAMPLE NUMBER	ELEV. (FEET)	NATURAL MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	PENETRATION* RESISTANCE
S-1	586.9	18.9	-	3-3-2
LS-1	581.9	-	-	4-5-7
LS-2	576.9	33.1	86.8	2-2-3
LS-3	571.9	-	-	2-2-4
VS-1	566.9	-	-	
LS-4	561.9	18.6	111.6	2-3-4
LS-5	556.9	-	-	2-3-4
LS-6	551.9	-	-	2-3-3
LS-7	546.9	-	-	2-3-3
PS-1	541.4	21.3	108.4	PUSHED
LS-8	536.9	-	-	2-3-4
VS-2	531.9	-	-	
LS-9	526.9	22.8	102.6	2-3-4
LS-10	521.9	-	-	2-3-4
PS-2	516.4	24.6	103.5	PUSHED
LS-11	511.9	8.3	138.9	14-28-40

NOTES:

1. Borings advanced using 6-inch diameter solid-stem augers to 10 feet, and 3-7/8 inch diameter tricone roller bit with recirculating drilling fluid to bottom of hole. 4-inch diameter casing was driven to 12.5 feet.
2. Groundwater was encountered at 8.2 feet during drilling.
3. Boring backfilled with cement grout to surface.

TOTAL DEPTH: 80.0'
BORING STARTED: 12/26/84
BORING COMPLETED: 12/27/84
INSPECTOR: D. Vensel
DRILLER: J. Blank
CONTRACTOR: American Drilling Co.

WATER LEVEL IN HOLE AT INDICATED NUMBER OF HOURS AFTER COMPLETION OF BORING WITH 0 FEET OF CASING IN PLACE.

*PENETRATION RESISTANCE: NUMBER OF BLOWS REQUIRED TO DRIVE 2 INCH O.D. SOIL SAMPLER 12 INCHES, USING 140 POUND WEIGHT WITH 30 INCH FREE FALL.

NEYER, TISEO & HINDO, LTD.
CONSULTING ENGINEERS

LOG OF TEST BORING NUMBER TB-2

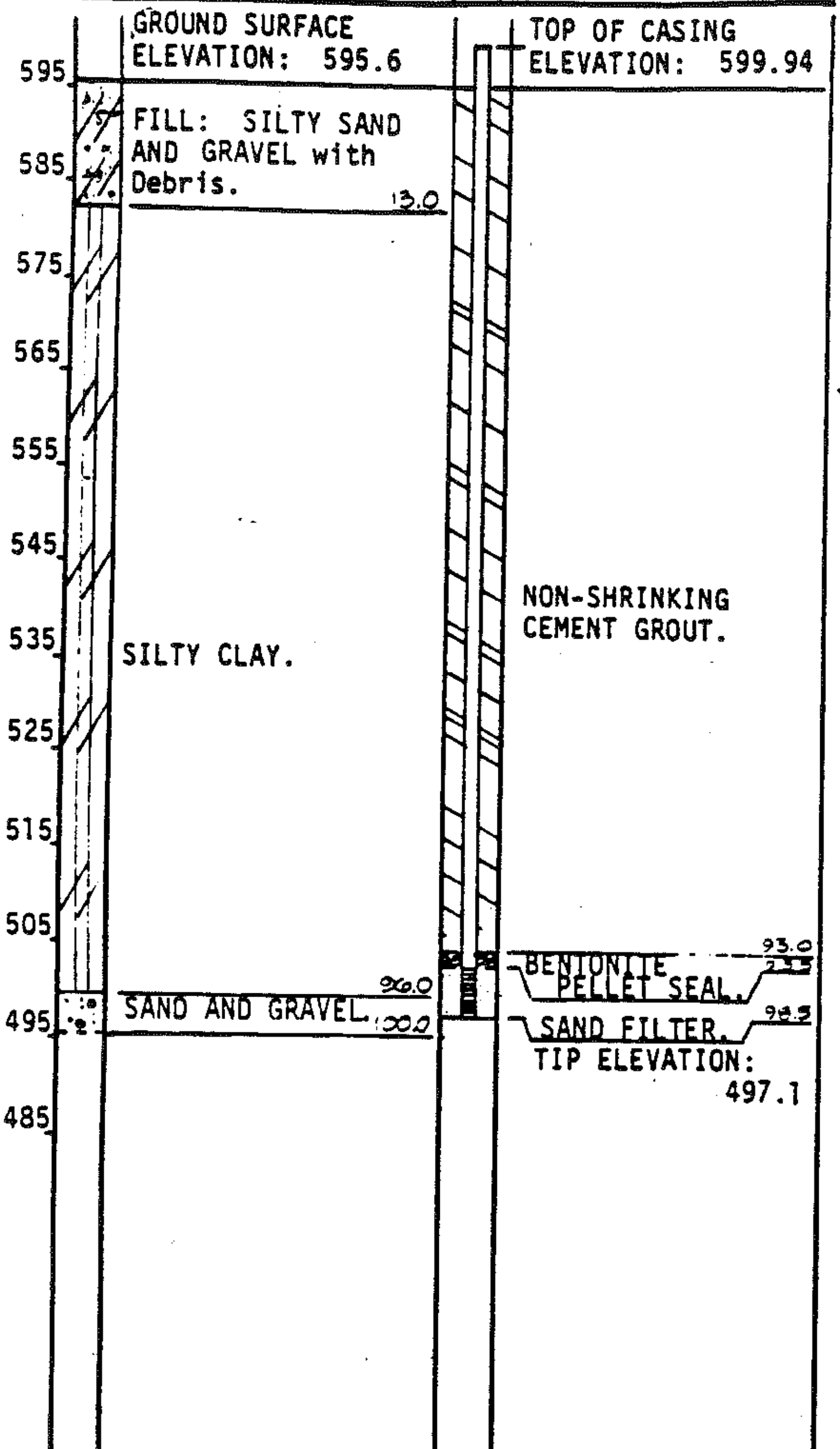
ALLEN PARK CLAY MINE LANDFILL
FORD MOTOR COMPANY
ALLEN PARK, MICHIGAN

APPROVED BY:	DATE: 1/11/85
PROJECT No. 84185 OW	FIGURE No. 2

LOG OF GROUNDWATER MONITORING WELL

CLASSIFICATIONS BY:
NEYER, TISEO & HINDO, LTD.

GENERALIZED SUBSURFACE PROFILE	WELL SCHEMATIC



NOTES:
1. Installed in test boring. For details of subsoil stratification, see Log of Test Boring TB-1.
2. Hole reamed with 5-3/4 inch diameter rotary-wash methods prior to well installation while 6-inch diameter steel casing was set at 12.5 feet.
3. Well annulus grouted with non-shrinking cement grout.

GROUNDWATER DATA

DATE	GROUND-WATER ELEV. (FEET)	COMMENTS
1/11/85	605.03	
1/16/85	604.93	Water froze in piezometric tube before reaching equilibrim.

CASING - DIAMETER: 2.0"
- LENGTH: 97.8'
- MATERIAL: Galvanized Steel

SCREEN - DIAMETER: 2.0"
- LENGTH: 5.0'
- MESH: #18 slot
- MATERIAL: Stainless Steel

WELL STARTED: 12-19-84
WELL COMPLETED: 12-26-84
INSPECTOR: L. Kendall/D. Vensel
DRILLER: J. Blank
CONTRACTOR: American Drilling Co
EQUIPMENT: CME-75

NOTES: (Continued)
4. Ground surface and casing top elevations provided by Wayne Disposal, Inc.

NEYER, TISEO & HINDO, LTD.
CONSULTING ENGINEERS
38000 TEN MILE RD., FARMINGTON HILLS, MI 48024

GROUNDWATER MONITORING WELL No. MW-1

ALLEN PARK CLAY MINE LANDFILL
FORD MOTOR COMPANY
ALLEN PARK, MICHIGAN

APPROVED BY:	DATE: 1-24-85
PROJECT NO: 84185	FIGURE NO: 3

PROJECT NO. 84185 CW	NEYER, TISEO & HINDO, LTD.	SHEET 1 OF 2

TABULATION OF TEST DATA

Test Boring or Test Pit Number	Sample Number	Depth of Sample Tip	Elevation of Sample Tip	Unconfined Compressive Strength (PSF)	Failure Strain (Percent)	Natural Water Content (Percent of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Volumetric Analysis: Solids (Percent)	Volumetric Analysis: Liquids (Percent)	Volumetric Analysis: Air (Percent)	Particle Size Distribution: Colloids (Percent)	Particle Size Distribution: Clay (Percent)	Particle Size Distribution: Silt (Percent)	Particle Size Distribution: Fine Sand (Percent)	Particle Size Distribution: Medium Sand (Percent)	Particle Size Distribution: Coarse Sand (Percent)	Particle Size Distribution: Gravel (Percent)	Atterberg Limits: Liquid Limit (Percent)	Atterberg Limits: Plastic Limit (Percent)	Atterberg Limits: Plasticity Index (Percent)	Apparent Specific Gravity
1	LS-1	5.0	590.6	-	-	12.4	101.4											-	-	-	
1	PS-1	17.5	578.1	1440	18.0	23.9	105.9											24	14	10	
1	LS-5	25.0	570.6	-	-	36.6	84.7											-	-	-	
1	PS-2	32.5	563.1	1280	20.0	17.9	116.4											25	14	11	
1	LS-7	45.0	550.1	-	-	21.1	107.9											-	-	-	
1	PS-3	60.5	535.1	1160	20.0	21.7	110.3											30	16	14	
1	LS-9	75.0	520.6	-	-	25.9	99.7											-	-	-	
1	LS-11	85.0	510.6	-	-	21.7	106.2											-	-	-	
1	LS-13	95.0	500.6	-	-	31.6	88.5											-	-	-	
2	S-1	5.0	586.9	-	-	18.9	-											-	-	-	
2	LS-2	15.0	576.9	-	-	33.1	86.8											-	-	-	
2	LS-4	30.0	561.9	-	-	18.6	111.6											-	-	-	
2	PS-1	50.5	541.4	1380	20.0	21.3	108.4	(See Consolidation Test Results)										30	16	14	
2	LS-9	65.0	526.9	-	-	22.8	102.6											-	-	-	

Figure 4

PROJECT NO. 84185 CW

NEYER, TISEO & HINDO, LTD.

TABULATION OF TEST DATA

Test Boring or Test Pit Number	Sample Number	Depth of Sample Tip	Elevation of Sample Tip	Unconfined Compressive Strength (PSF)	Failure Strain (Percent)	Natural Water Content (Percent of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Volumetric Analysis: Solids (Percent)	Volumetric Analysis: Liquids (Percent)	Volumetric Analysis: Air (Percent)	Particle Size Distribution: Colloids (Percent)	Particle Size Distribution: Clay (Percent)	Particle Size Distribution: Silt (Percent)	Particle Size Distribution: Fine Sand (Percent)	Particle Size Distribution: Medium Sand (Percent)	Particle Size Distribution: Coarse Sand (Percent)	Particle Size Distribution: Gravel (Percent)	Atterberg Limits: Liquid Limit (Percent)	Atterberg Limits: Plastic Limit (Percent)	Atterberg Limits: Plasticity Index (Percent)	Apparent Specific Gravity
2	PS-2	75.5	516.4	910	10.0	24.6	103.5											29	16	13	
2	LS-11	80.0	511.9	-	-	8.3	138.9											-	-	-	

Figure 5

Project No. 84185 NEYER, TISEO & HINDO, LTD.

Project Allen Park Clay Mine Landfill

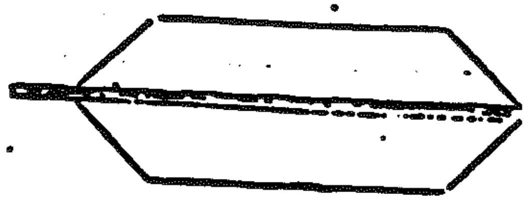
Location Allen Park, Michigan **DATE** December 19, 1984

FIELD VANE SHEAR TEST REPORT

TEST NO.	VS-1	ELEV. TOP OF HOLE	595.6
BORING HOLE NO.	TB-1	DEPTH TO TEST POINT	35.0
LINK & STA.	-	ELEV. OF TEST POINT (Tip of Vane)	560.6
OFFSET	-		

VANE DATA

TORQUE ARM LGTH. 12 IN.
TORQUE ARM DIA. 3/4 IN.
VANE LGTH. 5 IN.
VANE DIA. 2½ IN.



Ultimate Shear Strength (S) (Lbs./Sq. Ft.) = $\frac{30.87}{12}$ x Applied Torque (T) (In. - Lbs.)

FRICTION ON VANE SHAFT		UNDISTURBED CONDITION		REMOLDED CONDITION	
Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)
		5	23	5	14.0
		10	28.5	10	15.5
		12	29	15	15.5
		15	28	20	15.5
		20	27.5	25	15.5
		25	28.5		
		30	28		
		35	27		

READINGS & CALCULATIONS	UNDISTURBED CONDITION	REMOLDED CONDITION
Maximum Force Gage Reading for Vane (Lbs)	29.0	15.5
Maximum Force Gage Reading for Shaft (Lbs)	-	-
Net Force (Lbs)	29.0	15.5
Applied Torque (T) = Net Force x Torque Arm (In-Lbs)	348.0	186.0
Ultimate Shear Strength (S) = $\frac{1}{Kr}$ T	895	478

Sensivity = $\frac{\text{Shear Strength (Undisturbed)}}{\text{Shear Strength (Remolded)}}$ = 1.87

Natural Water Content 17.9 % (Sample PS-2) Half-Unconfined Comprehensive Strength 640 Lbs./Sq. Ft.

TECHNICIAN LK CHECKED BLF

Comments Increase shear strength by 7% to 960 psf based on Bjerrum's connection factor (Ref.1).

Ref 1: Bjerrum, L. "Embankments on Soft Ground", Proceedings of the Specialty Conference on Performance of Earth and Earth Supported Structures, ASCE, Vol. 2, 1972. pp.1-54.

Figure 6

Project No. 84185

NEYER, TISEO & HINDO, LTD.

Project Allen Park Clay Mine Landfill

Location Allen Park, Michigan

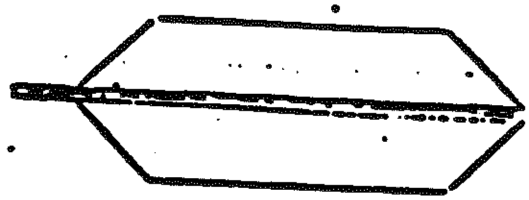
DATE December 20, 1984

FIELD VANE SHEAR TEST REPORT

TEST NO.	VS-2	ELEV. TOP OF HOLE	595.6
BORING HOLE NO.	TB-1	DEPTH TO TEST POINT	55.0
LINK & STA.	-	ELEV. OF TEST POINT (Tip of Vane)	540.6
OFFSET	-		

TORQUE ARM LGTH. 12 IN.
TORQUE ARM DIA. 3/4 IN.
VANE LGTH. 5 IN.
VANE DIA. 2.5 IN.

VANE DATA



Ultimate Shear Strength (S) (Lbs./Sq. Ft.) = $\frac{30.87}{12}$ x Applied Torque (T) (In. - Lbs.)

FRICTION ON VANE SHAFT		UNDISTURBED CONDITION		REMOLDED CONDITION	
Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)
		5	2	5	9
		10	15.5	10	10
		15	25	15	11
		20	28	20	11.2
		25	27	25	12
		30	25	30	12
				35	12.5
				40	12

Remo deg.
45
50
55

READINGS & CALCULATIONS	UNDISTURBED CONDITION	REMOLDED CONDITION
Maximum Force Gage Reading for Vane (Lbs)	28.0	12.5
Maximum Force Gage Reading for Shaft (Lbs)	-	-
Net Force (Lbs)	28.0	12.5
Applied Torque (T) = Net Force x Torque Arm (In-Lbs)	336.0	150.0
Ultimate Shear Strength (S) = $\frac{1}{AF}$ T	864	385

Sensivity = $\frac{\text{Shear Strength (Undisturbed)}}{\text{Shear Strength (Remolded)}}$ = 2.24

Natural Water Content 21.7% (Sample PS-3) Half-Unconfined Comprehensive Strength 580 Lbs./Sq. Ft.

TECHNICIAN LK CHECKED BLF

Comments Increase shear strength by 3% to 890 psf based on Bjerrum's correction factor. (Ref.1).

Project No. 84185 NEYER, TISEO & HINDO, LTD.

Project Allen Park Clay Mine Landfill

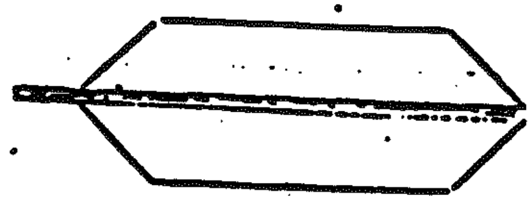
Location Allen Park, Michigan DATE December 20, 1984

FIELD VANE SHEAR TEST REPORT

TEST NO.	VS-3	ELEV. TOP OF HOLE	595.6
BORING HOLE NO.	TB-1	DEPTH TO TEST POINT	70.0
LINE & STA.	-	ELEV. OF TEST POINT (Tip of Vane)	525.6
OFFSET	-		

VANE DATA

TORQUE ARM LGTH. 12 IN.
TORQUE ARM DIA. 3/4 IN.
VANE LGTH. 5 IN.
VANE DIA. 2.5 IN.



Ultimate Shear Strength (S) (Lbs./Sq. Ft.) = $\frac{30.87}{12}$ x Applied Torque (T) (In. - Lbs.)

FRICTION ON VANE SHAFT		UNDISTURBED CONDITION		REMOLDED CONDITION	
Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)
-	-	5	1.5	5	6.0
		10	5.0	10	7.0
		15	12.0	15	7.0
		20	19.0	20	7.0
		25	19.0	25	6.5
		30	18.5	30	6.0
		35	18.0		

READINGS & CALCULATIONS	UNDISTURBED CONDITION	REMOLDED CONDITION
Maximum Force Gage Reading for Vane (Lbs)	19.0	7.0
Maximum Force Gage Reading for Shaft (Lbs)	-	-
Net Force (Lbs)	19.0	7.0
Applied Torque (T) = Net Force x Torque Arm (In-Lbs)	228.0	84.0
Ultimate Shear Strength (S) = $\frac{1}{AF}$ T	586	216

Sensivity = $\frac{\text{Shear Strength (Undisturbed)}}{\text{Shear Strength (Remolded)}}$ = 2.71

Natural Water Content - %   Half-Unconfined Comprehensive Strength - Lbs./Sq. Ft.

TECHNICIAN LK   CHECKED BLF

Comments Increase shear strength by 3% to 600 psf, based on Bjerrum's correctional factor (Ref.1)

Figure 8

Project No. 84185 NEYER, TISEO & HINDO, LTD.

Project Allen Park Clay Mine Landfill

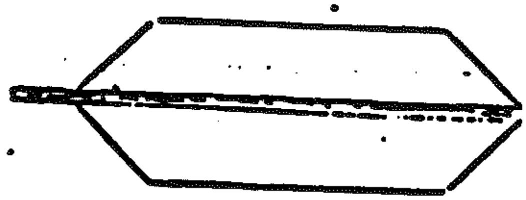
Location Allen Park, Michigan DATE December 26, 1984

FIELD VANE SHEAR TEST REPORT

TEST NO.	VS-1	ELEV. TOP OF HOLE	591.9
BORING HOLE NO.	TB-2	DEPTH TO TEST POINT	25.0
LINK & STA.	-	ELEV. OF TEST POINT (Tip of Vane)	566.9
OFFSET	-		

TORQUE ARM LGTH. 12 IN.
TORQUE ARM DIA. 3/4 IN.
VANE LGTH. 5 IN.
VANE DIA. 2½ IN.

VANE DATA



Ultimate Shear Strength (S) (Lbs./Sq. Ft.) = $\frac{30.87}{12}$ x Applied Torque (T) (In. - Lbs.)

UNDISTURBED CONDITION		UNDISTURBED CONDITION		REMOLDED CONDITION	
Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)
0	0	35 40	33	5	8
5 10	0 0	45	33 34	10	9
15 20	2 12.5	50 55	34 34	15	9
25 30	22 30				

READINGS & CALCULATIONS	UNDISTURBED CONDITION	REMOLDED CONDITION
Maximum Force Gage Reading for Vane (Lbs)	34	9
Maximum Force Gage Reading for Shaft (Lbs)	-	-
Net Force (Lbs)	34	9
Applied Torque (T) = Net Force x Torque Arm (In-Lbs)	408	108
Ultimate Shear Strength (S) = $\frac{1}{Kr}$ T	1050	278

Sensivity = $\frac{\text{Shear Strength (Undisturbed)}}{\text{Shear Strength (Remolded)}}$ = 3.78

Natural Water Content - % Half-Unconfined Comprehensive Strength - Lbs./Sq. Ft.

TECHNICIAN D.R.V. CHECKED BLF

Comments Increase shear strength by 7% to 1120 based on Bjerrum's correction factor (Ref.1).

Figure 9

Project No. 54785 NEYER, TISEO & HINDO, LTD.

Project Allen Park Clay Mine Landfill

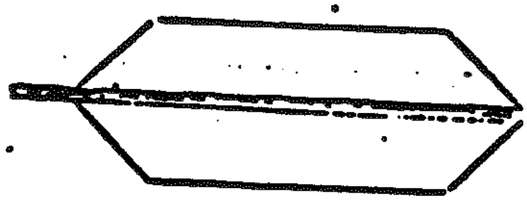
Location Allen Park, Michigan **DATE** December 27, 1984

FIELD VANE SHEAR TEST REPORT

TEST NO.	VS-2	ELEV. TOP OF HOLE	591.9
BORING HOLE NO.	TB-2	DEPTH TO TEST POINT	60.0
LINK & STA.	-	ELEV. OF TEST POINT (Tip of Vane)	531.9
OFFSET	-		

VANE DATA

TORQUE ARM LGTH. 12 IN.
TORQUE ARM DIA. 3/4 IN.
VANE LGTH. 5 IN.
VANE DIA. 2½ IN.



Ultimate Shear Strength (S) (Lbs./Sq. Ft.) = $\frac{30.87}{12}$ x Applied Torque (T) (In. - Lbs.)

FRICTION ON VANE SHAFT		UNDISTURBED CONDITION		REMOLDED CONDITION	
Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)	Rotation (Degrees)	Force Gage Reading (Lbs)
		0	0		
		5	18	0	0
		10	20		
		15	20.5	5	11.2
		20	20	10	11.5
		25	19.5	15	11.5
		30	19.5	20	11.5

READINGS & CALCULATIONS	UNDISTURBED CONDITION	REMOLDED CONDITION
Maximum Force Gage Reading for Vane (Lbs)	20.5	11.5
Maximum Force Gage Reading for Shaft (Lbs)	-	-
Net Force (Lbs)	20.5	11.5
Applied Torque (T) = Net Force x Torque Arm (In-Lbs)	246	138
Ultimate Shear Strength (S) = $\frac{1}{Ar}$ T	633	355

Sensivity = $\frac{\text{Shear Strength (Undisturbed)}}{\text{Shear Strength (Remolded)}}$ = 1.78

Natural Water Content - % Half-Unconfined Comprehensive Strength - Lbs./Sq. Ft.

TECHNICIAN D.R.V. CHECKED BLF

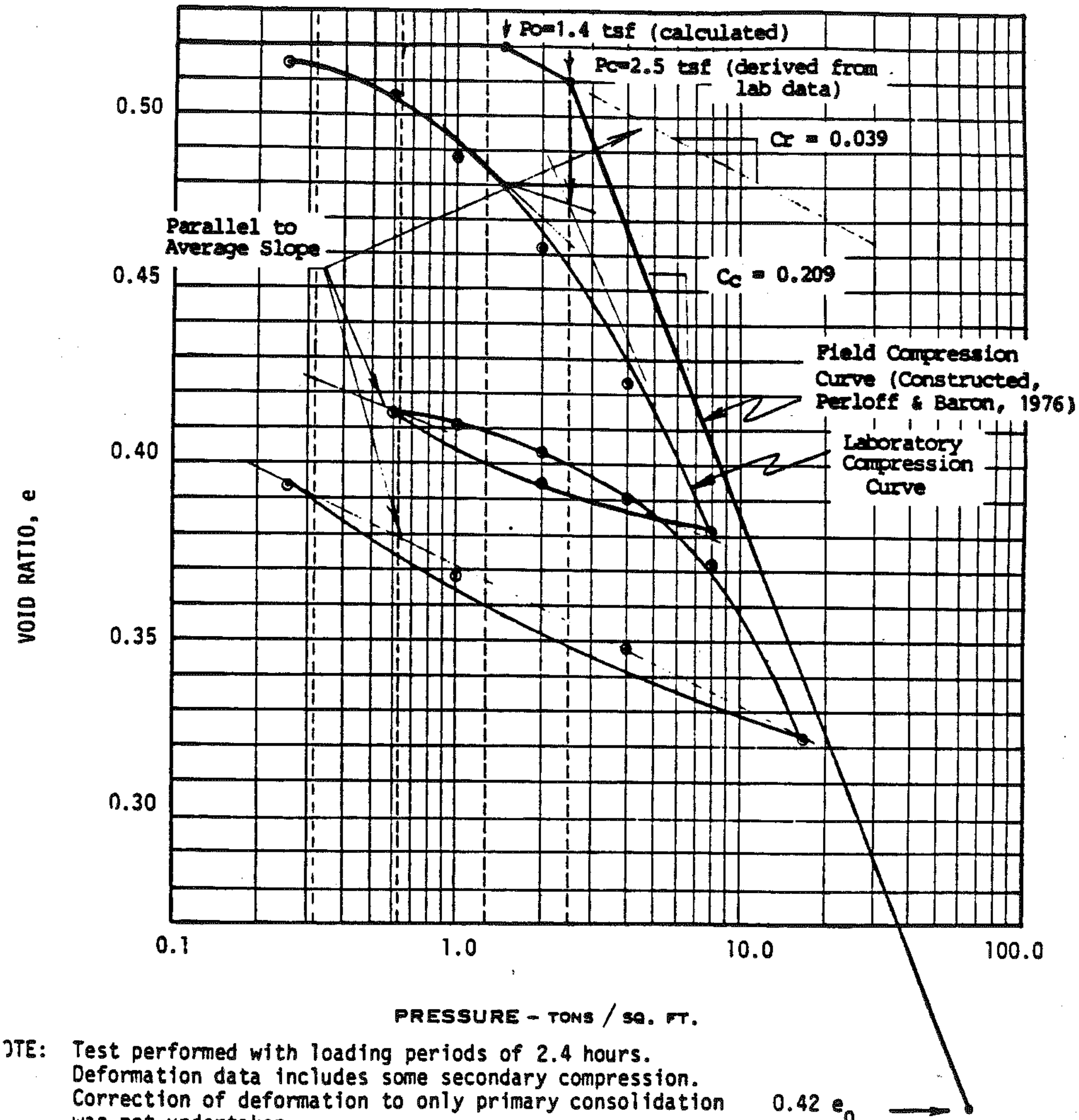
Comments Increase shear strength by 3% to 650 psf based on Bjerrum's correction factor. (Ref.1)

Figure 10

NEYER, TISEO & HINDO, LTD.

CONSOLIDATION TEST

LAB SAMPLE # - PROJECT NO. 84185 OW SHEET 1 OF 1
FOR Allen Park Clay Mine Landfill DATE 1/17/85
PROJECT LOCATION Allen Park, Michigan TESTED BY S.Y.
BORING # TB-2 FIELD SAMPLE # PS-1 SAMPLE ELEV. (TIP) 541.4 CHECKED BY BLF
Sample Dia. 2.50 inches



NOTE: Test performed with loading periods of 2.4 hours. Deformation data includes some secondary compression. Correction of deformation to only primary consolidation was not undertaken.

Figure 11

APPENDIX II

LIST OF CALCULATIONS

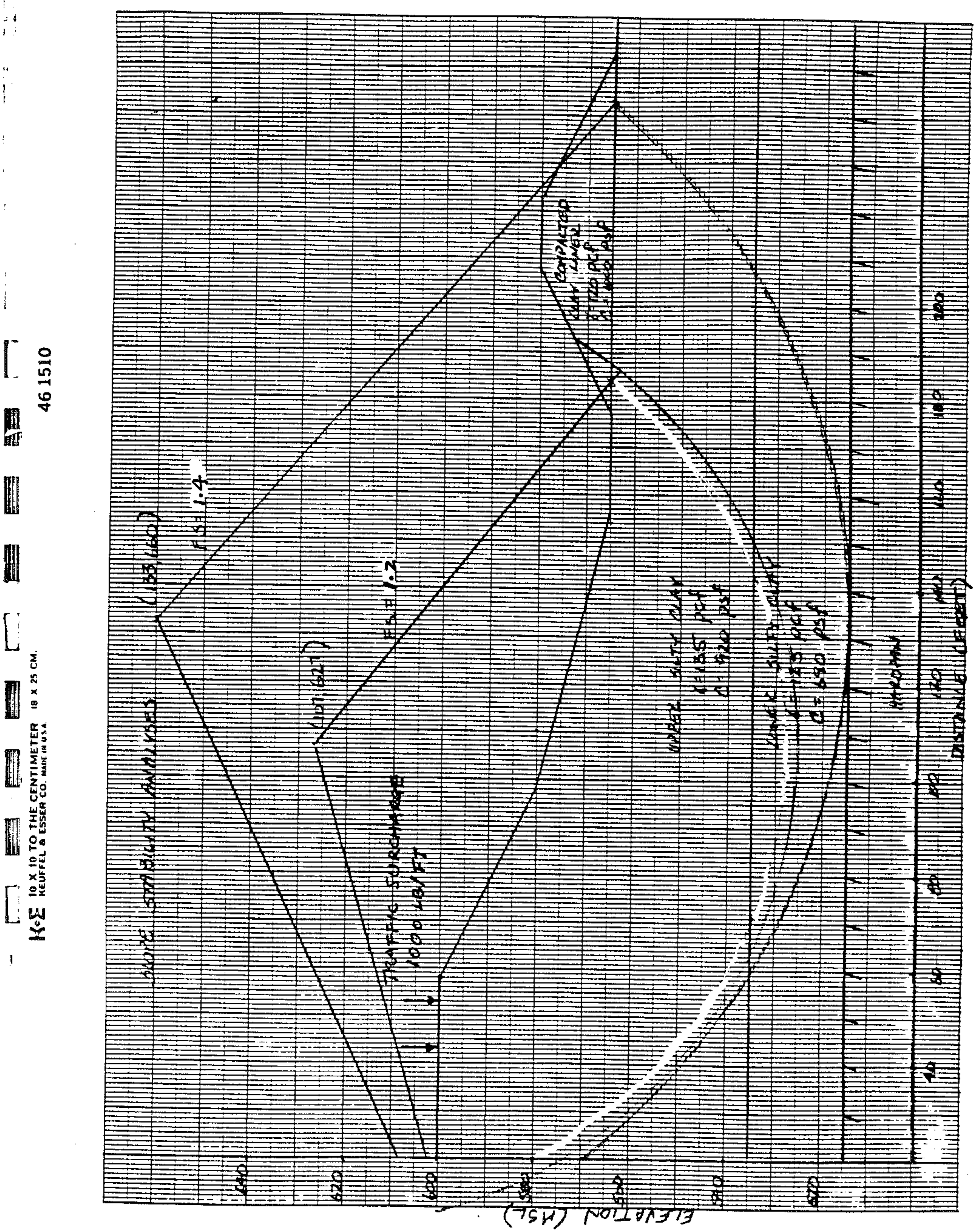

K+E 10 X 10 TO THE CENTIMETER 18 X 25 CM.
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 1510
SLOPE STABILITY ANALYSES
FS = 1.4
FS = 1.2
TRAFFIC SURCHARGE
1000 LB/FT
UPPER SILTY CLAY
LOWER SILTY CLAY
CLAY COMPACTED LINER
DISTANCE (FEET)
ELEVATION (MSL)

FIGURE 1

NEYER, TISEO & HINDO, LTD.
CONSULTING ENGINEERS
30999 Ten Mile Road • Farmington Hills, MI 48024 • (313) 471-0750
2053 South Dort Highway • Flint, MI 48503 • (313) 232-9652
2615 Comerica Building • Detroit, MI 48226 • (313) 965-0036

JOB A.P. Clay Mine PROJECT NO. 84185 SHEET NO. 9/20
SUBJECT Basal Heave BY JK DATE 1/22/85
CHK. BY BLF DATE 1/22/85
REVISED BLF 2/13/85

Check basal heave due to hydrostatic pressure. Based on the piezometric levels presented on the log of Monitoring Well #1, the maximum piezometric level in the vicinity of landfill Cell #2 is assumed to be at Elev. 605. Where clay extends below this elevation, excavation may also extend a critical distance such that the remaining clay balances the piezometric head with a safety factor of 1.2. This critical distance must be determined.

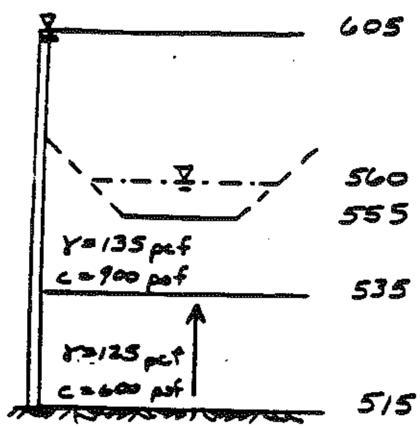
Excavation currently extends to El. 555, with ponded water to El 560. Based on the NTH Supplemental Subsurface Investigation, the clay extends to El. 515 (minimum). Therefore, determine the existing safety factor against basal heave, the safety factor for the construction (dewatered) condition, and the required excavation elevation for long-term stability. Use total stress analysis.

NOTE: DESIGN PLANS WERE MODIFIED SUBSEQUENT TO THIS ANALYSIS INCORPORATING THE RECOMMENDATIONS PRESENTED HEREIN REGARDING BASAL STABILITY.
BLF 2/13/85

FIGURE 2

NEYER, TISEO & HINDO, LTD.
CONSULTING ENGINEERS
30999 Ten Mile Road • Farmington Hills, MI 48024 • (313) 471-0750
2053 South Dort Highway • Flint, MI 48503 • (313) 232-9652
2615 Comerica Building • Detroit, MI 48226 • (313) 965-0036

JOB: A.P. Clay Mine — PROJECT NO. 84185 — SHEET NO. 10/20
SUBJECT: Basal Heave — BY: J.L. — DATE: 1/22/85
CHK. BY: BLF — DATE: 1/22/85



EXISTING CONDITION: $5(62.4) + 20(135) + 20(125) = 5512$ psf → 5616

$90(62.4) = 5616$ psf

$FS = \frac{5512}{5616} = 0.98 \rightarrow \underline{1.0}$

However, this situation appears stable in the field. Therefore, assume FS = 1.0, and the weight of soil = 5616 psf

CONSTRUCTION CONDITION: (DEWATERED) $5616\text{ psf} - 5(62.4\text{ psf}) = 5304$ psf

$FS = \frac{5304}{5616} = \underline{0.94}$

Based on this analysis, it appears that removal of the ponded water may result in basal heave in localized areas.

FIGURE 3

NEYER, TISEO & HINDO, LTD.
CONSULTING ENGINEERS
30999 Ten Mile Road • Farmington Hills, MI 48024 • (313) 471-0750
2053 South Dort Highway • Flint, MI 48503 • (313) 232-9652
2615 Comerica Building • Detroit, MI 48226 • (313) 965-0036

JOB A.P. Clay Mine PROJECT NO. 84185 SHEET NO. 11/20
SUBJECT Basal Heave BY JK DATE 1/22/85
CHK. BY BLF DATE 1/22/85

Now, calculate the required depth of fill to balance the hydrostatic pressure with a safety factor of 1.2.

$(605-515)\,62.4\text{ pcf }(1.2) = 6739.2\text{ psf}$

$6739.2 - 5304 = 1435.2\text{ psf}$

weight of existing soil (saturated)

Assume: clay fill placed above El. 555 will have a saturated unit weight of ~135 pcf (similar to in-situ).

So, $\dfrac{1435.2\text{ psf}}{135\text{ pcf}} = 10.6\text{ ft}$ say 11 ft of fill

Therefore, the required elevation of clay to adequately balance the hydrostatic pressure is El. 555 + 11 = El. 566.

If the minimum cell elevation is raised to El. 567, the safety factor > 1.2.

FIGURE 4

NEYER, TISEO & HINDO, LTD.
CONSULTING ENGINEERS
30999 Ten Mile Road • Farmington Hills, MI 48024 • (313) 471-0750
2053 South Dort Highway • Flint, MI 48503 • (313) 232-9652
2615 Comerica Building • Detroit, MI 48226 • (313) 965-0036

JOB Rouge Steel Clay Mine PROJECT NO. 84185 SHEET NO. 13/20
SUBJECT Settlement / Bearing Capacity: Profile BY BLF DATE 1/17
CHK. BY [illegible] DATE 1/22/85

GENERALIZED PROFILE - CELL II

ELEV.

610 (APPROXIMATE ORIGINAL GROUND EL.)

605 ▽

WASTE $\gamma_T = 75$ pcf

567

557 (APPROXIMATE BASE OF MANHOLE)

MEDIUM CLAY

① PI = 10 ; LL = 24 ; e_o = 0.51 ; c = 900 psf

P_c = 6000 psf ; C_c = 0.126 ; C_r = 0.019 (= 0.15 C_c)

γ = 135 pcf

535

SOFT TO MEDIUM CLAY

② PI = 14 ; LL = 30 ; e_o = 0.52 ; c = 600 psf

P_c = 5000 psf ; C_c = 0.21 ; C_r = 0.039

γ = 135 pcf

515

HARDPAN (INCOMPRESSIBLE)

FIGURE 5

CONSOLIDATION OF CLAY UNDER WEIGHT OF WASTE

SOIL PROPERTIES & PROFILE AS SHOWN IN FIGURE 13. PRECONSOLIDATION PRESSURES, C_c & C_r VALUES FOR THE MEDIUM CLAY ARE BASED ON THE FOLLOWING CORRELATIONS:

1) $\frac{C}{P_c} = 0.11 + 0.0037\,PI$

2) $C_c = 0.009\,(LL - 10)$

3) $C_r = 0.15\,C_c$

PRECONSOLIDATION PRESSURES, C_c & C_r VALUES FOR THE UNDERLYING SOFTER SOIL WERE DETERMINED FROM CONSOLIDATION TEST DATA, FIG. 11. THE HARDPAN IS ASSUMED TO BE INCOMPRESSIBLE.

THE WASTE IS ASSUMED TO HAVE AN AVERAGE DENSITY OF 75 pcf & AS SHOWN IN PREVIOUS CALCULATIONS DATED 6/28/84, LOSES 7% OF ITS VOLUME IN SECONDARY COMPRESSION. PRIMARY COMPRESSION WILL OCCUR DURING FILLING.

FIGURE 6

SETTLEMENT OF MANHOLE (AFTER NAVFAC DM 7.1)

ASSUME THAT THE EXCAVATION FOR THE MANHOLE IS OPEN LONG ENOUGH FOR REBOUND TO HAVE OCCURRED.

MANHOLE BASE IS 11 FT IN DIAMETER

LOWER SECTION HAS 10 FT O.D. & 9 FT. I.D. & IS 10 FT HIGH; UPPER SECTION HAS 6 FT O.D. & 5 FT. I.D. & IS 50 FT. IN LENGTH.

MANHOLE SUMP CONTAINS A MAXIMUM OF 1130 GAL. OR 3 FT HEIGHT OF FLUID. LEACHATE IS ASSUMED TO HAVE A SPECIFIC GRAVITY OF 1.03.

∴ PRESSURE EXERTED ON BASE FROM FULL LENGTH OF MANHOLE + 3' OF FLUID:

$$P = \frac{1}{\frac{\pi(11)^2}{4}}\left[150\frac{lb}{ft^3}\cdot\frac{\pi}{4}\left[(10^2-9^2)\cdot 10' + (6^2-5^2)\cdot 50'\right] + 64\frac{lb}{ft^3}\cdot\frac{\pi}{4}\cdot 9^2\cdot 3'\right]$$

$P = 1046$ psf

SOIL 1: $e_o = 0.91$ $C_c = 0.126$ $C_r = 0.019$

ESTIMATE SETTLEMENT OF CENTER OF MANHOLE:

LAYER	H	P_o	z	[1] I	ΔP	[2] P+ΔP	P_c	$S = \frac{C_r}{1+e_o}\cdot H\log\left(\frac{P+\Delta P}{P}\right)$	ΣS (in)
1	5'	181.5	2.5	1.0	1046	1227.5	6000	0.63"	0.63
2	5'	544.5	7.5	0.52	544	1088.5	6000	0.23"	0.86
3	5	907.5	12.5	0.29	261	1168.5	6000	0.08	0.94
4	5'	1270.5	17.5	0.14	146	1416.5	6000	0.03	0.97

FIGURE 7

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JOB Rouge Steel Clay Mine PROJECT NO. 84185 SHEET NO. 16/20
SUBJECT Settlement Analysis BY BLF DATE 1/18/85
CHK. BY JZ DATE 1/22/85

MAXIMUM ESTIMATED SETTLEMENT OF MANHOLE IS 1 INCH IF NO WASTE WERE PLACED

NOTES:
1. INFLUENCE VALUES FROM NAVFAC DM 7.1, CH. 4, FIG. 5.
2. PRESSURE INCREASES DO NOT EXCEED CALCULATED $P_c \Rightarrow C_r$ IS USED

SETTLEMENT OF CLAY UNDER THE WASTE (AFTER NAVFAC DM 7.1)

I. CONSOLIDATION:

ASSUME THAT THE LOADED AREA IS SUFFICIENTLY LARGE WITH RESPECT TO THE THICKNESS OF THE COMPRESSIBLE STRATUM FOR THE LOADING TO BE ONE-DIMENSIONAL, i.e. THE LOAD DOES NOT DISSIPATE WITH DEPTH.

LOAD PLACED ON COMPRESSIBLE STRATA EQUALS THE PRESSURE EXERTED BY 65 FT. OF WASTE & 5 FT OF CLAY COVER:

$$\Delta P = 65\,ft\,(75\,\tfrac{lb}{ft^3}) + 5\,ft\,(135\,\tfrac{lb}{ft^3}) = 5550\ pcf$$

EFFECTIVE STRESS ANALYSIS OF EXISTING CONDITIONS:

@ EL 515 $u = 62.4\,pcf\,(605-515) = 5616\ psf$
@ EL 567 $u = 0\ psf$ (ATMOSPHERIC)

∴ EFFECTIVE PORE PRESSURE DISTRIBUTION $= \dfrac{u_{515} - u_{567}}{567-515} = 108\ \dfrac{psf}{ft.\,depth}$

∴ EFFECTIVE UNIT WEIGHT OF SOIL = 135 − 108 pcf = 27 pcf

LAYER	H	Z	$P_o = \gamma' z$	$P_o + \Delta P$	P_c	$\frac{C_c}{1+e_o}$	$\frac{C_r}{1+e_o}$	$S = \frac{C_{c,r}}{1+e_o} \times H \log\left(\frac{P_o+\Delta P}{P_{c,o}}\right)$ (FT)
1	32	16	432	5982	6000	0.083		
							0.012	0.438
2	20	42	1134	6684	5000	0.138		0.348
							0.026	0.401

Σ = 1.2 FT

THE ELASTIC SETTLEMENT OF THE SOIL MUST ALSO BE CONSIDERED IN EVALUATING TOTAL SETTLEMENT.

FIGURE 8

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II. ELASTIC COMPRESSION:

AGAIN ASSUME THAT THE LOADING IS ONE-DIMENSIONAL. HARDPAN-LIKE SOILS WILL NOT SETTLE.

$$\epsilon = \frac{\sigma}{\underbrace{E}_{\text{YOUNG'S MODULUS}}} \qquad S_c = \epsilon H = \frac{\overset{\text{PRESSURE IMPOSED ON CLAY}}{\sigma}}{E}\underbrace{H}_{\text{THICKNESS}}$$

$$E_{clay} \approx 200\, q_u \approx 400\, c = 400\left[\overbrace{\frac{32 \times 900 + 600 \times 20}{55}}^{c_{average}}\right]$$

$$= 300 \text{ ksf} \qquad \text{(PERLOFF \& BARON, 1976)}$$

∴ THE ELASTIC SETTLEMENT:

$$S_c = \frac{5.33 \text{ ksf}}{300 \text{ ksf}}(55') = \underline{1.0 \text{ FT}}$$

III TOTAL SETTLEMENT OF CLAY LAYER UNDER THE WEIGHT OF THE WASTE:

SKEMPTON & BJERRUM (1957) SUGGEST A FACTOR OF 0.8 BE APPLIED TO THE PRIMARY CONSOLIDATION ESTIMATE FOR NORMALLY CONSOLIDATED TO MODERATELY OVERCONSOLIDATED CLAYS SUCH AS THOSE ON THIS SITE. THE TOTAL SETTLEMENT IS THE SUM OF THE ADJUSTED PRIMARY CONSOLIDATION & ELASTIC SETTLEMENT ESTIMATES. THEREFORE:

$$\text{TOTAL ESTIMATED SETTLEMENT} = 0.8(1.2 \text{ FT}) + 1 \text{ FT}$$

$$= 2.0 \text{ FT AT THE CENTER}$$

FIGURE 9

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JOB: ROUGE STEEL CLAY MINE PROJECT NO. 84183 SHEET NO. 18/20
SUBJECT: SETTLEMENT OF SIDE SLOPE BETWEEN CELLS I & II BY BLF DATE 1/17
CHK. BY ZY DATE 1/22/85

SEE PREVIOUS CALCULATIONS PERFORMED FOR COVER EVALUATION (6/28/84)

ESTIMATE OF POST-CLOSURE SETTLEMENT:

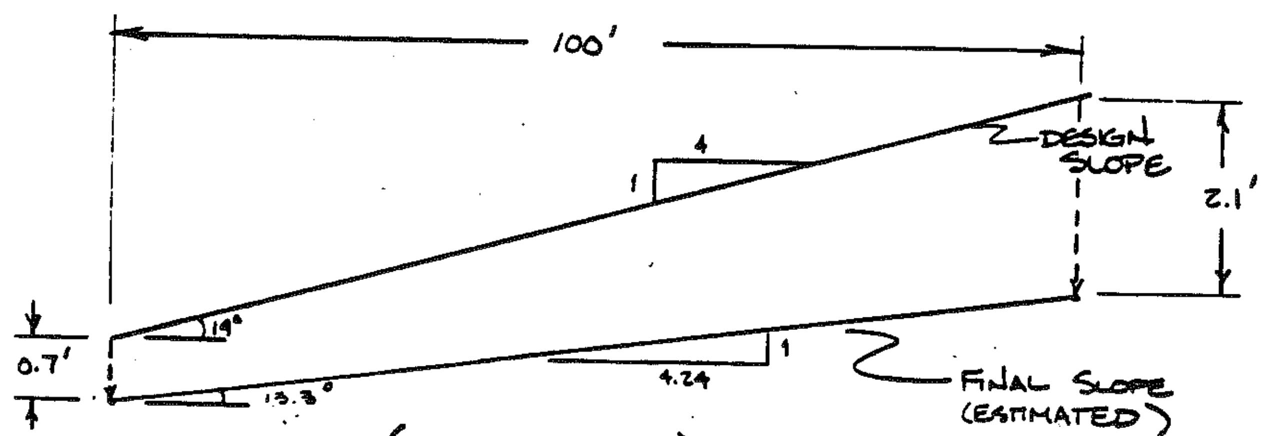
$$\frac{\Delta H}{H} \approx 7\%$$

FILL DEPTH AT TOP OF SLOPE = 30'

30 FT × 0.07 = 2.1 FEET

FILL DEPTH AT BOTTOM OF SLOPE = 10'

10 FT × 0.07 = 0.7 FEET



(NOT TO SCALE)

ESTIMATED CHANGE IN SLOPE DUE TO SETTLEMENT OF WASTE:

$$\Delta\phi = 0.7°$$

FIGURE 10

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JOB: Range Steel, Clay Mine PROJECT NO. 85189 SHEET NO. 19/20
SUBJECT: Bearing Capacity BY BLF DATE 1/18/85
CHK. BY [illegible] DATE 1/22/85

SOIL PROFILE & PARAMETERS ARE AS PREVIOUSLY SHOWN. BASE OF MANHOLE IS AT APPROXIMATELY ELEV. 557. MINIMUM EMBEDMENT IS 3 FT & ASSUME EXCAVATED MATERIAL IS USED AS FILL. THE RESULTANT BEARING VALUE IS WITH NO WASTE IN PLACE

$$\therefore q_{ult} = cN_c + \gamma D_f = 900(6.16) + 135(3')$$
$$= 5950 \text{ psf}$$

WITH A FACTOR OF SAFETY OF 3:

$q_{all} \approx 2000$ psf WITH 3' OF FILL

MAT FOR MANHOLE IS 8" THICK $\Rightarrow \frac{8''}{12''/ft} \times 150$ pcf $= 100$ psf FROM MAT

$\therefore$ NET ALLOWABLE PRESSURE IS 1900 psf

THE AREA OF THE MAT IS $\frac{\pi}{4}(11)^2 = 95 \text{ ft}^2$

$\therefore$ NET LOAD ALLOWABLE = 180 K

MANHOLE WEIGHT IS $150 \frac{lb}{ft^3} \times \frac{\pi}{4}(10^2 - 9^2) ft^2 = 2.2$ K/FT HT.

ASSUME SUMP IS FULL (3 FT OF LEACHATE) & 20 FT OF MANHOLE IS FREE STANDING.

$\therefore$ MAXIMUM LOAD:

$$Q = 2.2 K/ft \times 20 ft + \frac{64}{1000} kcf \times 3' \times 9^2 ft^2 = 56 K < 180 K$$

FIGURE 11

MANHOLE WILL BE RAISED CONCURRENTLY WITH FILL – NO MORE THAN 10–15 FEET OF FREE STANDING MANHOLE WILL BE INSTALLED AHEAD OF THE FILL. AS FILL IS PLACED AROUND THE MANHOLE, THE BEARING CAPACITY WILL RISE.

REFERENCES:

1. NATIONAL SANITATION FOUNDATION, STANDARD 54 FOR FLEXIBLE MEMBRANE LINERS, NSF, ANN ARBOR, MI (1983)
2. NAVAL FACILITIES ENGINEERING COMMAND DESIGN MANUAL, NAVFAC DM 7.1, SOIL MECHANICS, ALEXANDRIA, VA, MAY 1982.
3. PERLOFF, W.H. & W. BARON, SOIL MECHANICS: PRINCIPLES AND APPLICATIONS, THE RONALD PRESS CO, NEW YORK, 1976, 745 PP.
4. SKEMPTON, A.W. & L. BJERRUM, "A CONTRIBUTION TO THE SETTLEMENT ANALYSIS OF FOUNDATIONS ON CLAY", GEO-TECHNIQUE, VOL. 7, NO. 3, P. 168, 1957.

FIGURE 12

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211 Willowbrook Lane, West Chester, Pennsylvania 19382 215 436-0632

JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO. 1
SUBJECT DESIGN OF WATER PRESSURE RELIEF SYSTEM (BENEATH SECONDARY LINER) BY BS DATE 3-3-88
CHK. BY CJM DATE 3-7-88

BACKGROUND

ACCORDING TO THE FINDINGS OF A HYDROGEOLOGIC INVESTIGATION CONDUCTED BY NTH IN EARLY 1985, THERE IS AN UPWARD HYDRAULIC FLOW GRADIENT AT THE SITE UNDERLYING THE PROPOSED CELL II. THE DRAINAGE SYSTEM PROPOSED BELOW IS TO MINIMIZE ANY SIGNIFICANT BUILD-UP OF WATER PRESSURE UNDERLYING THE SECONDARY LINER (FML), WHICH MAY IN TURN ADVERSELY AFFECT THE INTEGRITY OF THE PRIMARY CLAY LINER ABOVE.

PROPOSED DRAINAGE SYSTEM

THE PROPOSED DRAINAGE SYSTEM GENERALLY COMPRISES 5 FT STRIPS OF GEOSYNTHETIC DRAINS SPACED AT 50 FT CENTERS. AS SHOWN ON SHEET NO. 2, ALL DRAINS ARE LED TO COLLECTION PIPES WHICH IN TURN ALLOW THE CAPTURED WATER TO FLOW TO A SUMP OUTSIDE THE CELL AREA FOR DISPOSAL.

THE GEOSYNTHETIC DRAINS ARE TO BE PLACED BETWEEN THE FML AND THE UNDERLYING SUBBASE CLAY. (REFER TO SHEET NO. 6). BETWEEN THE DRAINAGE NET AND THE CLAY IS A SHEET OF GEOSYNTHETIC FILTER FABRIC WHICH PREVENTS THE MIGRATION OF CLAY PARTICLES INTO THE DRAINS.

FIGURE 13

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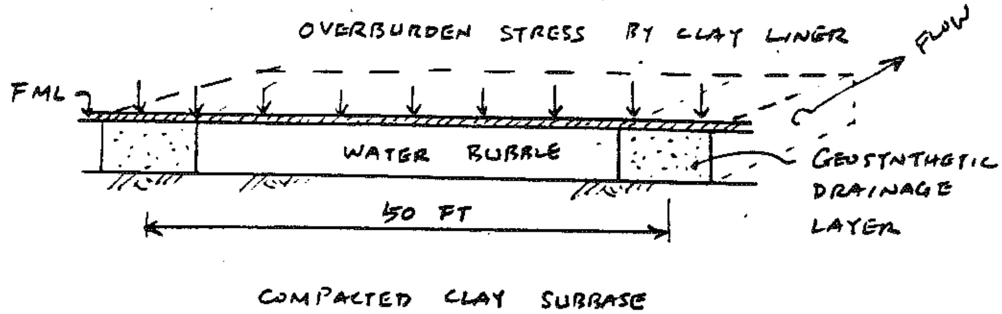
JOB: ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO. 3
SUBJECT: DESIGN OF WATER PRESSURE RELIEF SYSTEM (BENEATH SECONDARY LINER) BY TBS DATE 3-2-88
CHK. BY GJM DATE 3-7-88

EVALUATE THE EFFICIENCY OF DRAINAGE NET



ASSUME THAT THE CLAY SUBBASE IS SATURATED. AS THE PORE WATER PRESSURE AT THE CELL BASE CAN BE GREATER THAN THE OVERBURDEN PRESSURE EXERTED BY THE CLAY LINER, A BUBBLE OF WATER CAN BE TRAPPED AS SHOWN ABOVE.

TO PREVENT THE WATER BUBBLE FROM EXPANDING, TRANSMISSIVITY OF THE DRAINAGE LAYER SHOULD BE SUFFICIENTLY HIGH SUCH THAT:

INFLOW DUE TO UPWARD GRADIENT $\ngtr$ OUTFLOW THROUGH DRAINS

CONSIDER 50 FT DRAIN SPACING.

FROM STABILITY OF CELL BASE CALCS. : $v = 8.27 \times 10^{-10}$ IN./SEC

$$\therefore \text{INFLOW RATE} = 8.27 \times 10^{-10} \frac{\text{IN}}{\text{SEC}} \times \frac{1}{12} \frac{\text{FT}}{\text{IN.}} \times 50 \text{ FT}$$
$$= 3.45 \times 10^{-9} \text{ FT}^2/\text{SEC}$$

AT DRAIN END (WHERE IT IS CONNECTED TO COLLECTION PIPE),

$$Q_{MAX} = 3.45 \times 10^{-9} \frac{\text{FT}^2}{\text{SEC}} \times \underbrace{200 \text{ FT}}_{\text{DRAIN LENGTH}}$$
$$= 6.89 \times 10^{-7} \text{ FT}^3/\text{SEC}$$

FIGURE 14

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JOB ALLEN PARK CLAY MINE PROJECT NO. 86347 OW SHEET NO. 4
SUBJECT DESIGN OF WATER PRESSURE RELIEF SYSTEM BY TSS DATE 3-3-88
CHK. BY CJM DATE 3-7-88

DETERMINE MINIMUM TRANSMISSIVITY OF DRAINAGE NET.

$$Q = k\,i\,A = k\,i\,(\underset{\text{THICKNESS}}{\overset{\text{WIDTH}}{B\,t}}) = \overset{\text{TRANSMISSIVITY}}{T}\,i\,B$$

$$\therefore \quad T = \frac{Q}{i\,B}$$

FOR PROPOSED DRAINAGE NETWORK,

$i \simeq 2\%$

$B \simeq 5$ FT (TYPICAL WIDTH)

$$\therefore \quad T = \frac{6.89 \times 10^{-7}}{0.02 \times 5} = 6.89 \times 10^{-6} \text{ FT}^2/\text{SEC}$$

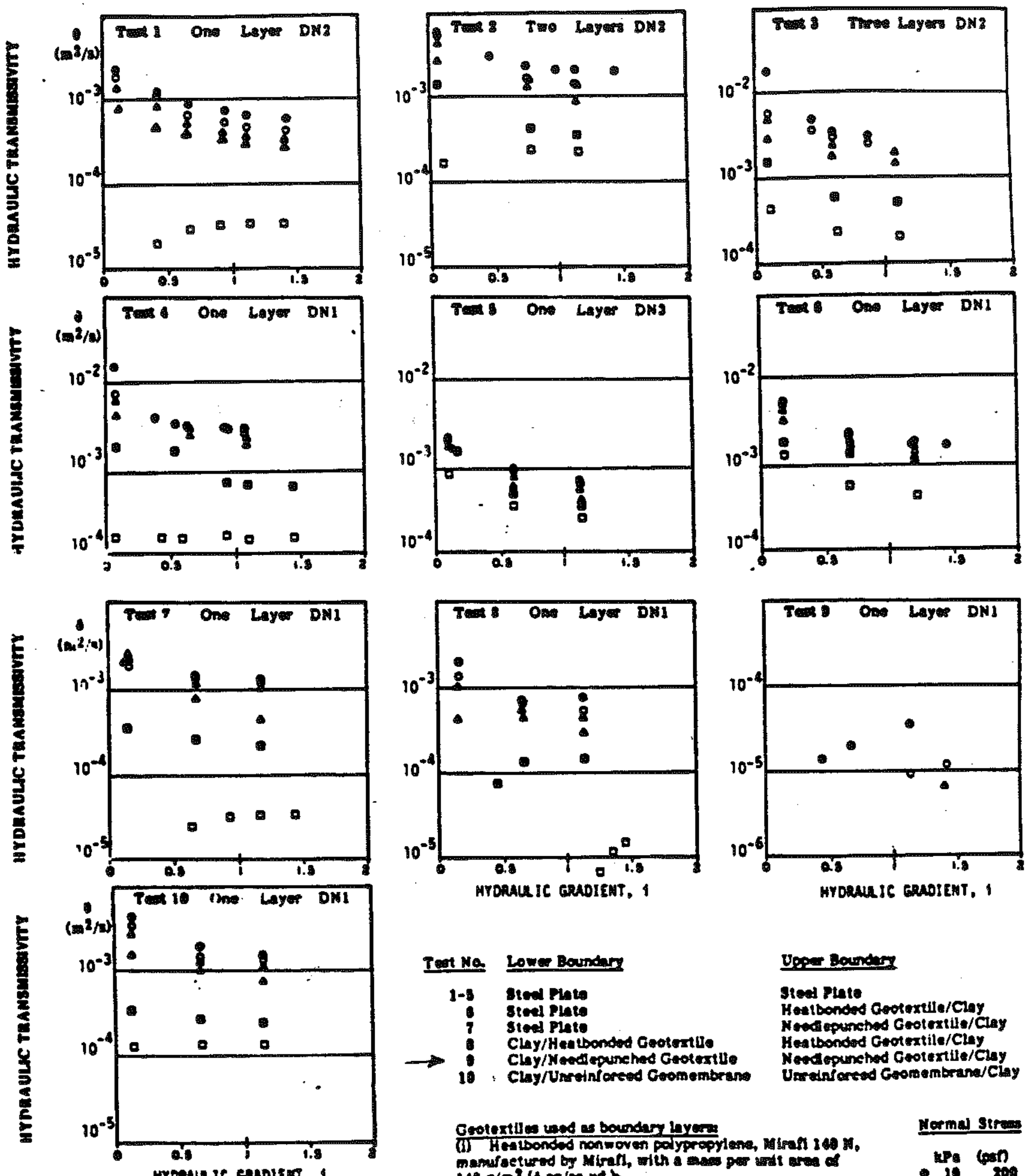
$$= 6.4 \times 10^{-7} \text{ M}^2/\text{SEC} \quad (\leftarrow \text{MIN. REQUIRED})$$

→ USE TENSAR™ DN1 OR EQUIVALENT AS DRAINAGE NET (REFER TO PUBLISHED TRANSMISSIVITY DATA ON SHEET NO. 5. NOTE THAT THE AVAILABLE TRANSMISSIVITY IS ON THE ORDER OF 10^{-5} M^2/SEC [TEST NO. 9] AND SHOULD THEREFORE BE ABLE TO MEET THE TRANSMISSIVITY REQUIREMENT)

FIGURE 15

REFERENCE: WILLIAMS, N., GIROUD, J.P. & BONAPARTE, R. "PROPERTIES OF PLASTIC NETS FOR LIQUID & GAS DRAINAGE ASSOCIATED WITH GEOMEMBRANES"

SHEET 5



Test No.	Lower Boundary	Upper Boundary
1-5	Steel Plate	Steel Plate
6	Steel Plate	Heatbonded Geotextile/Clay
7	Steel Plate	Needlepunched Geotextile/Clay
8	Clay/Heatbonded Geotextile	Heatbonded Geotextile/Clay
9	Clay/Needlepunched Geotextile	Needlepunched Geotextile/Clay
10	Clay/Unreinforced Geomembrane	Unreinforced Geomembrane/Clay

Geotextiles used as boundary layers:
(i) Heatbonded nonwoven polypropylene, Mirafi 140 N, manufactured by Mirafi, with a mass per unit area of 140 g/m² (4 oz/sq.yd.);
(ii) Needlepunched nonwoven polypropylene, Fibretex 400, manufactured by Crown Zellerbach with a mass per unit area of 400 g/m² (12 oz/sq.yd.).
Geomembrane used as a boundary layer:
Unreinforced 0.5 mm (20 mil) thick PVC.

Normal Stress

kPa	(psf)
10	200
50	1,000
100	2,000
200	4,000
350	7,000
500	10,000

Fig. 5 Results of transmissivity tests at 20°C on nets DN1, DN2 and DN3 performed under a range of normal stresses with various boundary conditions.

FIGURE 16

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JOB ALLEN PARK CLAY MINE PROJECT NO. 86347OW SHEET NO. 6
SUBJECT DESIGN OF WATER PRESSURE RELIEF SYSTEM BY ABS DATE 3-3-88
CHK. BY CJM DATE 3-7-88

CHOICE & EVALUATION OF GEOTEXTILE FILTER

GRAIN SIZE DISTRIBUTIONS OF TYPICAL BROWN SILTY CLAYS USED FOR THE CELL SUBBASE ARE SHOWN ON SHEET NOS. 7 & 8.

FROM THESE CURVES, $0.04 \le d_{85} \le 0.075$ mm.

MORE THAN 50% OF MATERIAL IS FINER THAN #200 SIEVE SIZE.

ACCORDING TO FEDERAL HIGHWAY ADMINISTRATION (FHA)* FILTRATION CRITERION,

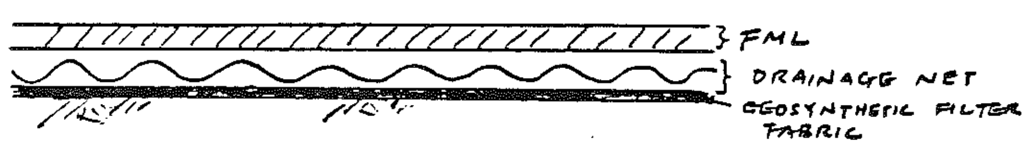
$$\underset{(\#70)}{0.211 \text{ mm}} > EOS > \underset{(\#100)}{0.149 \text{ mm}}$$

WHERE EOS = EQUIVALENT OPENING SIZE

→ RECOMMENDATION:

USE FIBRETEX® 400 OR TREVIRA® 1127 OR EQUIVALENT NON-WOVEN NEEDLE-PUNCHED GEOTEXTILE FABRICS (REFER TO SHEET NO. 9 & 10 FOR PROPERTIES OF FABRICS)

CLAY LINER



CLAY SUBBASE

* REFERENCE: ROLLIN, A.L. & DENIS, R., "GEOSYNTHETIC FILTRATION IN LANDFILL DESIGN", PROCEEDINGS, GEOSYNTHETIC '87 CONFERENCE, NEW ORLEANS, PP 456-470

FIGURE 17

Trevira® Spunbonds are highly needled non-woven fabrics with excellent tensile properties, high filtration potential and outstanding permeability.

Trevira® Spunbonds are 100% polyester continuous spun needlepunched engineering fabrics. They deliver a combination of advantages unmatched by any other spunbonded geotextiles. They're resistant to freeze-thaw, soil chemicals and ultraviolet light exposure—excellent where the requirement is (1) tensile reinforcement, (2) planar flow, (3) filtration, and (4) separation. For example, in roadways, railbeds, drainage systems, pond liners, retaining walls. And much more. Trevira® Spunbonds are extraordinary engineering fabrics.

TYPICAL PHYSICAL PROPERTIES OF TREVIRA® TYPE 11 PRODUCTS

Fabric Type	**1115**	**1120**	**1127**	**1135**	**1145**	**1155**
Fabric Weight (oz/yd^2)	4.5	6	8	10	13	16
Thickness (mils) (ASTM D-1777)	85	100	125	150	175	210
Grab Strength (LB, MD/CD*) (ASTM D-1682)	130/110	175/155	260/225	340/300	430/390	525/485
Grab Elongation, (%, MD/CD) (ASTM D-1682)	85/95	85/95	85/90	90/95	90/95	90/95
Trapezoid Tear Strength (LB,MD/CD) (ASTMD-1117)	50/45	65/60	100/95	130/130	185/180	205/200
Puncture Strength—5/16" (LB) (ASTM D-751)	60	90	125	155	200	260
Mullen Burst Strength (PSI) (ASTM D-3786)	220	300	380	500	600	800
Vertical Water Flow ($GAL/MIN/FT^2$) (HFI Test)	325	300	280	265	240	220
EOS (CW-02215)	70^+	50-70	70-100	70^+-100^+	100^+-120^+	120^+
Std. Roll Widths (FT)	12.5, 14.5 and 16.0					
Std. Roll Length (FT)	300 and 1000				300 and 600	

*MD = Machine Direction, CD = Cross Machine Direction

Special width and length rolls are available upon request.

Note: Typical Physical Properties of Type 11 Products represent typical average values as opposed to specification values. For recommended end use specifications and physical properties contact your TREVIRA Spunbond Distributor.

FIGURE 18

VERSATILE

SHEET 10

FABRIC GRADES

For pond applications, Fibretex is supplied in four grade thicknesses. Roll sizes are 13.0 or 17.5 ft. wide by 300 ft. long.

Custom sizes can be ordered to meet the special requirements of any pond project. The availability of these different grades and sizes permits the most economical choice for each application (see Fabric Grade Selector Guide).

FABRIC GRADE SELECTOR GUIDE

FOR CONTAINMENT APPLICATIONS

Grade (Thickness)	200 (60 mils)	300 (90 mils)	400 (110 mils)	600 (150 mils)
GAS VENTING		●	●	●
SLOPE STABILITY	●	●	●	
SOIL DRAINAGE	●	●	●	●
SOIL SEPARATION	●	●		
SLUDGE DEWATERING	●	●	●	
GEOMEMBRANE PUNCTURE PROTECTION	●	●	●	●
GEOMEMBRANE SEPARATION			●	●
GEOMEMBRANE ABRASION		●	●	●
SOLID WASTE LAND FILL COVER	●	●	●	●

FIBRETEX GEOTEXTILE FABRICS TYPICAL FABRIC PHYSICAL PROPERTIES

FABRIC PROPERTY	200 GRADE	300 GRADE	✓ 400 GRADE	600 GRADE
Thickness (mils) ASTM D-1777	60	90	110	150
Grab strength (lbs.) ASTM D-1682	140	210	260	320
Elongation (%) ASTM D-1682	125	140	150	150
Trapezoid tear strength (lbs.) ASTM D-2263	60	75	100	120
Burst strength (psi) ASTM D-751	250	300	400	500
Water permeability, k (cm/sec.)	.30	.30	.30	.30
Equivalent opening size (U.S. standard sieve)	70-100	80-100	80-100	80-100
pH Resistance Range	1-13	1-13	1-13	1-13

ROLL DATA*

Standard widths (ft.)	13 & 17.5	13 & 17.5	13 & 17.5	13
Standard length (ft.)	300	300	300	300
Weight (lbs.)	170 & 230	230 & 320	320 & 425	450

*Custom grades and widths up to 18 ft. are available upon request.

FIGURE 19

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JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO. 11
SUBJECT DESIGN OF WATER PRESSURE RELIEF SYSTEM BY PS DATE 3-3-88
CHK. BY CM DATE 3-7-88

DESIGN OF COLLECTION PIPES

COMPUTE SITE AREA:

$\frac{1}{2} \times 500 \times (620+650) = 317{,}500 \ FT^2$

$\frac{1}{2} \times 120 \times (75+80) = 9{,}300 \ FT^2$

$\frac{1}{2} \times 230 \times (120+270) = 44{,}850 \ FT^2$

$240 \times 270 = 64{,}800 \ FT^2$

TOTAL $= 436{,}450 \ FT^2$

INFLOW DUE TO UPWARD GRADIENT $= 8.27 \times 10^{-10}$ IN/SEC

$\therefore$ MAX. FLOW IN COLLECTION PIPE

$$= 8.27 \times 10^{-10} \times 436{,}450 \ FT^2 \times 144 \ \frac{IN^2}{FT^2}$$

$$= 0.052 \ IN^3/SEC$$

MANNING'S EQUATION:

$$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$$, WHERE $R_h = \frac{1}{2} R$ ($\leftarrow$ ASSUME FLOW FULL)

BUT $S = 1\%$ FROM DRAWING (MINIMUM)

$n = 0.015 \leftarrow$ ASSUMED

$$\therefore \quad \frac{0.052}{\pi R^2} = \frac{1.49}{0.015} \left(\frac{R}{2}\right)^{2/3} (0.01)^{1/2}$$

$$0.00265 = R^{2/3+2} = R^{2.67}$$

$\therefore$ $R = 0.108$ IN $\Rightarrow$ DIAMETER REQ'D $= 0.217$ IN

$\rightarrow$ USE A 4" ϕ HDPE PIPE
SLOPED AT 1% MIN.

FIGURE 20

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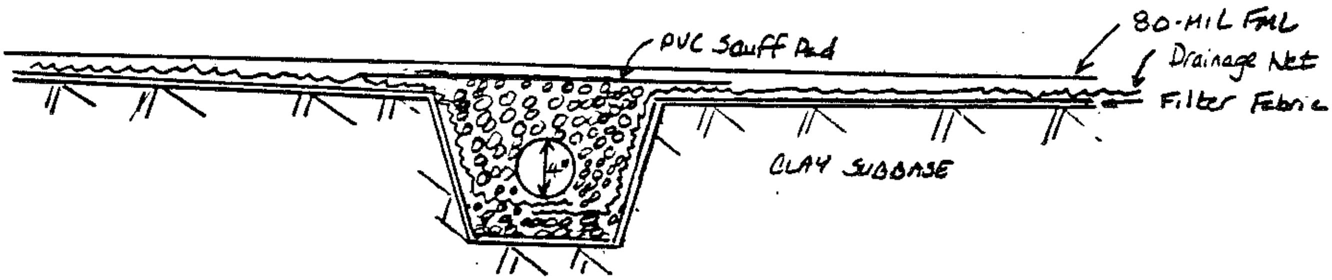
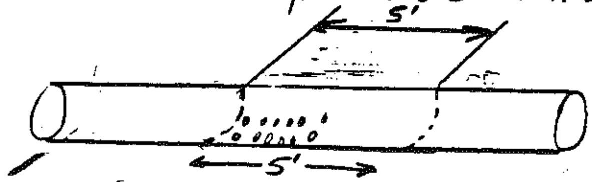
JOB Allen Park Clay Mine PROJECT NO. 863470W SHEET NO. 1
SUBJECT Proposed seepage Collection System BY CTM DATE 3-7-98
CHK. BY DATE

Design of seepage Collection System

It is anticipated that the main collector pipes will be placed in shallow trenches excavated in the subbase. The entire length of pipe will be surrounded by 4-6" inches of pea gravel. The collector pipe will be perforated at the location where the drainage wick joins the pipe. The drainage wick will extend across the base of the cell to the trench holding the main collector pipe. The wick will sit within the trench between the collector pipe invert and the pea gravel envelope.



Assume perforations extend for a distance of five feet where wick joins filter fabric. Assume that perforations consist of 2-rows of 1/4 inch ϕ holes spaced at 4-inch intervals along the pipe.



Determine that entrance velocity does not exceed 0.1 ft/s

$Q = VA$

$V = Q/A < 0.1\ ft/s$

FIGURE 21

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JOB: Atlas Park Clay Mine — PROJECT NO. 863470W — SHEET NO. 2
SUBJECT: Proposed Seepage Collection System — BY CTM — DATE 3-7-88
CHK. BY ____ DATE ____

A = Area of perforations = $\pi R^2 \cdot$ (Number of holes)

$$32 \text{ holes} \times \left[\left(\frac{1}{4}''\right)\left(\frac{1}{2}\right)\right]^2 \pi = 1.57 \text{ in}^2$$

$$= 0.011 \text{ ft}^2$$

Q = Flow rate out of 2 wick drains into pipe

$$= (6.89 \times 10^{-7} \text{ ft}^3/\text{s})(2)$$

$$= 1.38 \times 10^{-6} \frac{\text{ft}^3}{\text{s}}$$

$$V = 1.38 \times 10^{-6} \text{ ft}^3/\text{s} \Big/ 0.011 \text{ ft}^2 = 1.25 \times 10^{-4} \frac{\text{ft}}{\text{s}}$$

$$6.33 \times 10^{-6} \frac{\text{ft}}{\text{s}} \ll 0.1 \frac{\text{ft}}{\text{s}}^{*}, \quad \therefore \text{ OK}$$

<u>Check filter criteria of pipe/pea gravel interface</u>

For Circular Holes:

$$\frac{\text{Minimum filter } D_{85}}{\text{Hole diameter}} \geq 1.2 \qquad \text{(USEPA-SW 870)}$$

0.375 → diameter of pea gravel

$$0.375 / 0.25 = 1.5$$

$1.5 > 1.2$ $\therefore$ OK, ie, peastone shouldn't clog pipe.

* recommended entrance velocity for wells, <u>Groundwater & Wells</u>, Driscoll

FIGURE 22

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JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO.
SUBJECT PRESSURE RELIEF SYSTEM BY BS DATE 6-24-88
CHK. BY LK DATE 6/24/88

CHECK PIPE DEFLECTION

4" ϕ HDPE SDR21 PIPES MANUFACTURED BY PLEXCO HAS BEEN SPECIFIED FOR USE.

CHECK DEFLECTION USING SPANGLER'S EQUATION (REFER TO FIG. 36 FOR REFERENCE)

DETERMINE W:

VERTICAL PRESSURES ON PIPE, σ_v

γ_{WASTE} = 75 PCF × 20* FT = 1500

γ_{LINER} = 135 PCF × 5 FT = 675

$\gamma_{DRAIN\ (SAND)}$ = 120 PCF × 1 FT = 120

$\gamma_{DRAIN\ (GRAVEL)}$ = 130 PCF × 0.5 FT = 65

$\Sigma\sigma_v$ = 2360 PSF

* NOTE: REQUIRED WASTE THICKNESS TO COUNTERACT UPLIFTING FORCE DUE TO UPWARD GROUNDWATER FLOW

PIEZ. ELEV. AT CELL BOTTOM

$$= \frac{(590-560)\,62.4 - (125 \times 5)}{75} = 17 \text{ FT}, \text{ SAY } 20 \text{ FT}$$

TO ACCOUNT FOR PERFORATIONS IN PIPE,

$$\lambda_p = \tfrac{1}{4}"\phi/\text{HOLE} \times 6 \text{ HOLES/FT} = 1.5 \text{ IN/FT}$$

NEGLECTING THE TRENCH EFFECT, WHICH COULD REDUCE σ_v.

$$(\sigma_v)_{DESIGN} = \frac{12}{12-1.5} \times 2360 = 2697 \text{ PSF}$$

FIGURE 22A

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JOB ALLEN PARK CLAY MINE PROJECT NO. 86347 OW SHEET NO.
SUBJECT PRESSURE RELIEF SYSTEM BY BS DATE 6-24-88
CHK. BY LK DATE 6/24/88

$B_c = 4.5$ IN.

$\therefore\ W = B_c \sigma_v = 4.5 \text{ IN} \times 2697 \frac{LB}{FT^2} \times \frac{1}{144} \frac{FT^2}{IN^2} = 84.3$ LB/IN

$r = \frac{B_c}{2} = 2.25$ IN

$E^* = 133{,}000$ PSI FOR PE 3408

$E' = 2250$ PSI FOR PEA GRAVEL

$I = \frac{t^3}{12} = \frac{(0.215)^3}{12} = 8.28 \times 10^{-4}\ IN^3$

$$\therefore\ \Delta y = \frac{De\ K\ W\ r^3}{EI + 0.061\ E'\ r^3}$$

$$= \frac{1.5\ (0.10)\ (84.3)\ (2.25)^3}{133000\ (8.28 \times 10^{-4}) + 0.061\ (2250)(2.25)^3}$$

$$= 0.086 \text{ IN.}$$

$\frac{\Delta y}{B_c} = \frac{0.086}{4.5} = 0.019 \simeq 1.9\% < 7.5\%$ ← MAX. SUGGESTED BY MANUFACTURER

FIGURE 22B

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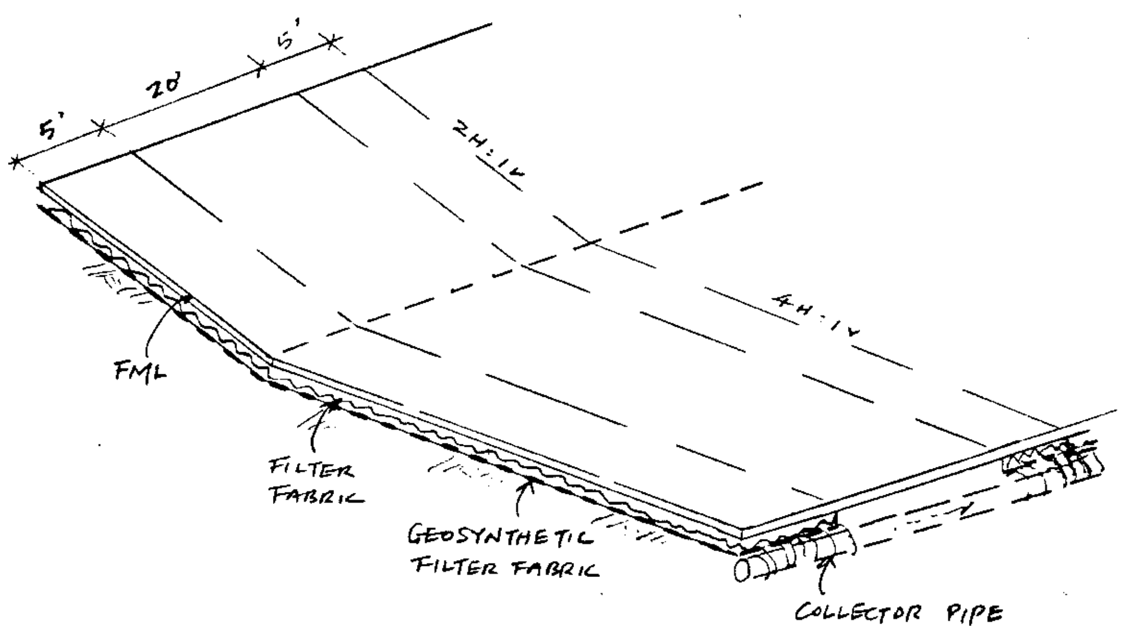
JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO.
SUBJECT DESIGN OF WATER PRESSURE RELIEF SYSTEM ON SIDESLOPES BY BS DATE 6-23-88
CHK. BY LK DATE 6/24/88

BACKGROUND:

AS SHOWN ON FIGURES 28 & 29, THE 5-FT CLAY LINER ON THE SIDESLOPES SHOULD THEORETICALLY BE STABLE EVEN THOUGH PORE WATER PRESSURE MAY BE DEVELOPED BELOW THE FML AS A RESULT OF UPWARD GROUNDWATER FLOW BELOW THE SITE.

DURING THE CONSTRUCTION STAGE, HOWEVER, EXCESS PORE WATER PRESSURE MAY DEVELOP BELOW THE FML UPON PLACEMENT OF THE 5-FT CLAY LINER. THE MAGNITUDE OF THIS EXCESS PORE WATER PRESSURE IS DEPENDENT ON THE DEGREE OF SATURATION OF THE NATIVE CLAY UNDERLYING THE FML.

TO MINIMIZE THE BUILD-UP OF EXCESS PORE WATER PRESSURE, WICK DRAINS ARE PROPOSED TO BE INSTALLED BELOW THE FML AS SHOWN ON MCI'S DRAWINGS. THE WICK DRAINS ARE LED TO COLLECTOR PIPES LOCATED AT THE TOE OF THE SLOPES. (SEE SKETCH BELOW)



SKEMATIC DRAWING OF WICK DRAINS ON SIDESLOPES

FIGURE 22C

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JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO.
SUBJECT DESIGN OF WATER PRESSURE RELIEF SYSTEM ON SIDESLOPES BY ABS DATE 6-24-88
CHK. BY LK DATE 6/24/88

ASSUME THAT ON-SITE NATIVE CLAY IS FULLY SATURATED (WORST CASE SCENARIO)

$$U_{MAX} = 5 \text{ FT} \times 130 \frac{LB}{FT^3} = 650 \text{ PSF.}$$

FROM CONSOLIDATION TEST RESULTS (FOR UNDERLYING CLAY),

$$C_r = 0.039 \quad ; \quad C_v = 1.2 \times 10^{-2} \text{ cm/sec.}$$

CONSIDER A 2-FT CLAY STRATUM UNDERLYING FML

$$t_{50} = \frac{0.197 \ (2 \times 12 \times 2.54)^2}{1.2 \times 10^{-2}} = 61006 \text{ SEC}$$

$$= 17 \text{ HRS}$$

$$t_{90} = 17 \times \frac{0.848}{0.197} = 73 \text{ HRS} \simeq 3 \text{ DAYS}$$

$$S = \frac{C_r}{1+e_0} H \log_{10} \left(\frac{P + \Delta P}{P} \right)$$

$$= \frac{0.039}{1+0.55} (2) \log \left(\frac{2 \times 135 + 5 \times 130}{2 \times 135} \right)$$

$= 0.027$ FT $= 0.32$ IN. ← TOTAL PRIMARY SETTLEMENT

∴ 50% CONSOLIDATION = 0.16 IN. ← SHOULD BE COMPLETE IN 17 HRS

CHECK TRANSMISSIVITY OF WICK DRAINS:

LENGTH OF DRAINS ON 1V:4H SLOPE

$$\text{RATE OF INFLOW} = \frac{0.5 \times 0.027 \times 25 \times 85}{61006}$$

$$= 4.70 \times 10^{-4} \text{ FT}^3/\text{SEC}$$

NOTE THAT THE SLOPE LENGTH ASSUMPTION IS CONSERVATIVE, PLACEMENT OF THE ENTIRE 1V:4H SLOPE IS ASSUMED TO BE COMPLETED IN ONE DAY

FIGURE 22D

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JOB: ALLEN PARK CLAY MINE PROJECT NO. 86347 OW SHEET NO.
SUBJECT: DESIGN OF WATER PRESSURE RELIEF SYSTEM ON SIDESLOPES BY ASS DATE 6-24-88
CHK. BY LK DATE 6/24/88

$$T = \frac{Q}{i B}$$

WHERE B = WIDTH OF WICK DRAINS $\simeq 5'$

$i = 1/4 = 0.25$

$$\therefore T = \frac{4.70 \times 10^{-4}}{0.25 \times 5} = 3.76 \times 10^{-4} \text{ FT}^2/\text{SEC}$$

$$= 3.49 \times 10^{-5} \text{ M}^2/\text{SEC} \leftarrow \text{REQUIRED}$$

THE REQUIRED TRANSMISSIVITY OF WICK DRAINS IS ON THE SAME ORDER OF MAGNITUDE PUBLISHED BY MANUFACTURER. (SEE FIG. 16)

ALSO, FROM RESULTS OF SOIL INVESTIGATION, THE IN-SITU NATIVE CLAY IS ONLY PARTIALLY SATURATED. THE ESTIMATED INFLOW RATE AND HENCE THE COMPUTED REQUIRED TRANSMISSIVITY IS EXPECTED TO BE OVER-ESTIMATED.

CONCLUSION:

THE PROPOSED WICK DRAIN SYSTEM ON THE SIDESLOPE SHOULD BE ABLE TO RELIEVE THE EXCESS PORE WATER PRESSURE THAT MAY DEVELOP UPON PLACEMENT OF THE 5-FT CLAY LINER

FIGURE 22E

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JOB A.P. Clay Mine PROJECT NO. 863470W SHEET NO. 1
SUBJECT Liner Evaluation BY CTM DATE 3-9-97
CHK. BY BS DATE 3/24/97

Check tensile stress on liner during installation
ie, hanging from the crest of the slope with no shear strength between liner and soil on slope

Assume 1:2 slope 23' high (46' long) + 1:4 slope, 17' high (68' long),
determine T stress for 80-mil HDPE, 60-mil HDPE, PN 3000 HDPE drainage net, 1 foot wide strip

Material Properties	* 80-mil	* 60-mil	** PN 3000
S.G.	0.94	0.94	0.936 g/cm³
Tensile Strength (lb/in-width)	140	120	53/31

* Standard 54, NSF 1985

** Fluid Systems Inc., product specifications

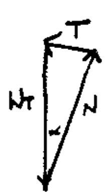
For 80 mil HDPE:

$$W_T = \left[(23^2+46^2)^{1/2} + ((17^2)+(68)^2)^{1/2}\right] \frac{(.08 \text{ in})}{(12 \text{ in/ft})} (62.4 \text{pcf})(0.94)$$

$$W_T = (51+70) \text{ feet } (.08 \text{ in}/12 \text{ in/ft})(62.4 \text{ pcf})(0.94)$$

$= 47.3$ lb/ft

$= 3.9$ lb/in along slope, for 1-inch wide strip



$$\alpha = \tan^{-1} 40/114 = 19.3°$$

$$T = W_T \sin 19° = 3.9 \frac{\text{lb}}{\text{in}} \times \sin 19° =$$

$= 1.3 \frac{\text{lb}}{\text{in}}$ along slope

FIGURE 23

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JOB A.P. Clay Mine PROJECT NO. 863470W SHEET NO. 2
SUBJECT BY CJM DATE 3-9-87
CHK. BY BS DATE 3/24/87

$1.3 \frac{lb}{in} \lll 140 \frac{lb}{in\ width}$, $\therefore$ 80-mil HDPE shouldn't fail in tension

For 60-mil HDPE

$$WT = (121\ ft)\left(\frac{.06\ in}{12\ in/ft}\right)(62.4\ pcf)(0.94)$$

$= 35.5\ lb/ft$

$= 3.0\ lb/in$ along slope

$T = WT \sin 19° = 3.0 \sin 19°$

$= 0.98\ lb/in$ along slope

$0.98 \frac{lb}{in} \lll 120$, $\therefore$ 60-mil HDPE shouldn't fail in tension

For PN 3000 Drainage Net*

$$WT = (121\ ft)\left(\frac{.200''}{(12\ in/ft)}\right)(0.936\ gm/cm^3) \times \left(\frac{62.43\ pcf}{gm/cm^3}\right)$$

$= 117.8\ lb/ft$

$= 9.8\ lb/in$ along slope

$T = (9.8\ lb/in) \sin 19°$

$= 3.2\ lb/in$ along slope

$3.2 \frac{lb}{in} \lll 53/31 \frac{lb}{in}$, $\therefore$ OKAY

* Assumed for use along slopes

FIGURE 24

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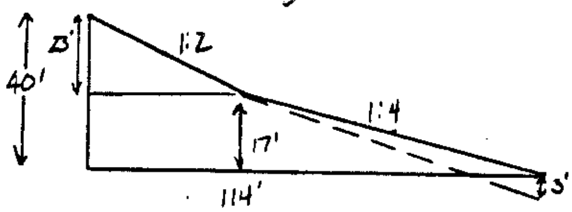
JOB [illegible] Park Clay Mine PROJECT NO. 863470W SHEET NO. 3
SUBJECT Liner Evaluation BY CM DATE 3-25-87
CHK. BY BS DATE 3/25/87

Check Required Elongation of Liner

① Case I - Assume that maximum settlement of 3 feet occurs at the toe of the slope.



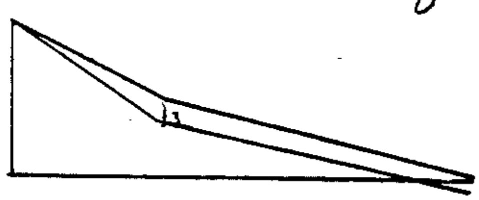
$$L_o = (23^2 + 46^2)^{1/2} + (17^2 + 68^2)^{1/2} = 51 + 70' = 121'$$

$$L_f = (23^2 + 46^2)^{1/2} + ((17+3)^2 + 68^2)^{1/2}$$

$$= 51.4 + 70.9 = 122.3'$$

$$\epsilon = \frac{122.3 - 121}{121} = 1.1\ \% \longrightarrow \text{OK}$$

② Case II - Assume that maximum settlement of 3 feet occurs at the end of the 1:2 slope.



$$L_o = 121'$$

$$L_f = ((23+3)^2 + 46^2)^{1/2} + (17^2 + 68^2)^{1/2}$$

$$= 122.93'$$

$$\epsilon = \frac{122.93 - 121}{121} = 1.6\% \longrightarrow \text{OK}$$

Material Properties	80-Mil	60-Mil	PN-3000
Elongation @ yield	10%	10%	
* Elongation @ Break	500%	500%	925/425%

* Supplied by manufacturer of PN-3000, however, another brand of drainage grid may be used

FIGURE 25

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Revised CJM 5/4/87
Checked by VBP 5/6/87

JOB Allen Park Clay Mine PROJECT NO. 863470W SHEET NO. 5
SUBJECT Leak Detection System Analysis BY CJM DATE 3-18-87
CHK. BY TBS DATE 3/25/87

Minimum Allowable Transmissivity of Leak Detection System

Assume Infiltration into leak detection system = 1/2 Infiltration into leachate collection system (conservative estimate)

$= 1/2\ (2.3 \times 10^{-6}\ cm/s)$

$= 1.15 \times 10^{-6}\ cm/s$

$= 3.78 \times 10^{-8}\ ft/s$

Determine Flow per Unit Width along 1:2 slope $\left(\frac{Q}{X} = Q \times L_{slope}\right)$

$\frac{Q}{X} = 3.78 \times 10^{-8}\ \frac{ft}{s} \times 39' = 1.48 \times 10^{-6}\ \frac{ft^2}{s}$ $\quad L = (18^2 + 35.1^2)^{1/2} = 39'$

$= 1.38 \times 10^{-3}\ \frac{cm^2}{s}$

$T = \frac{Q/X}{i} = \frac{1.38 \times 10^{-3}\ cm^2/s}{0.447}$ $\quad i = 1/(1^2 + 2^2)^{1/2}$

$= 3.09 \times 10^{-3}\ \frac{cm^2}{s}$, req'd T

Normal force that drainage fabric will be exposed to $= 5670\ psf + 5'(135\ pcf)$
5' compacted clay

$= 6345\ psf$

According to Manufacturer's info, For PN 4000* at 7000 psf and $i = 0.5$, $T = 1.5 \times 10^{-3}\ m^2/s$

$T = 1.5 \times 10^{-3}\ m^2/s$

$= 15\ cm^2/s$

$15\ \frac{cm^2}{s}$ @ 7000 psf $\gg 3.09 \times 10^{-3}\ cm^2/s$, $\therefore$ OK

* A brand other than PN-4000 may be used in the leak detection system

FIGURE 26

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Revised CJM 5/4/87
Checked By: VBP 5/6/87

JOB Allen Park Clay Mine PROJECT NO. 863470W SHEET NO. 6
SUBJECT Leak Detection System Analysis BY CJM DATE 3-18-87
CHK. BY VB DATE 3/24/87

Required T along base of Cell II
overall slope = 2.24%

$\frac{Q}{X} = Ti$ $\frac{Q}{X} =$ Infiltration x Maximum Length of Flow

From Preliminary Plans by MCI dated 4-20-87, maximum length of flow in leak detection system extends from SE corner of outer Cell base to southern sump = 295'

$$\frac{Q}{X} = \left(3.78 \times 10^{-8} \frac{ft}{s}\right)(295') = 1.12 \times 10^{-5} \frac{ft^2}{s}$$

$$T = \frac{Q/X}{i} = \frac{1.12 \times 10^{-5} \ ft^2/s}{.0224} = 5 \times 10^{-4} \frac{ft^2}{s}$$

$$= 0.46 \frac{cm^2}{s}$$

Manufacturer's specifications for PN 3000* lists $T = 0.0013 \frac{ft^2}{s} = 13 \frac{cm^2}{s}$ at 7000 psf

For 2.24% gradient

$$13 \frac{cm^2}{s} > 0.46 \frac{cm^2}{s} \quad \therefore \quad OK$$

* A brand other than PN-3000 may be used in the leak detection system

FIGURE 27

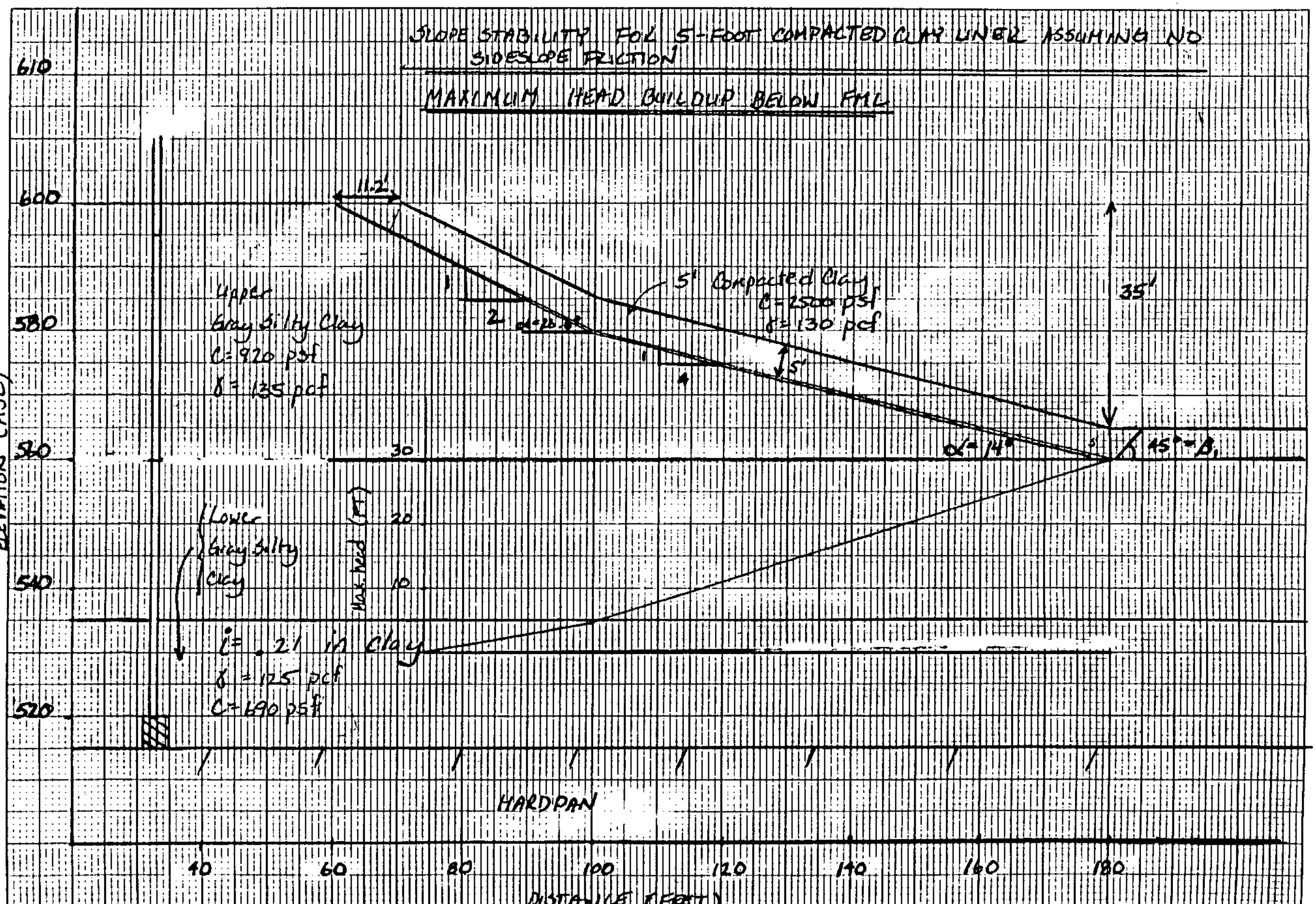
SLOPE STABILITY FOR 5-FOOT COMPACTED CLAY LINER ASSUMING NO SIDESLOPE FRICTION
MAXIMUM HEAD BUILDUP BELOW FML
11.2'
5' Compacted Clay
C=2500 psf
γ=130 pcf
35'
Upper
Gray Silty Clay
C=920 psf
γ=135 pcf
1
2
1
4
5'
α=14°
45°=β
Lower
Gray Silty
Clay
i=.21 in Clay
γ=125 pcf
C=690 psf
Max head (FT)
30
20
10
HARDPAN
ELEVATION (MSL)
610
600
580
560
540
520
40
60
80
100
120
140
160
180
DISTANCE (FEET)

FIGURE 28

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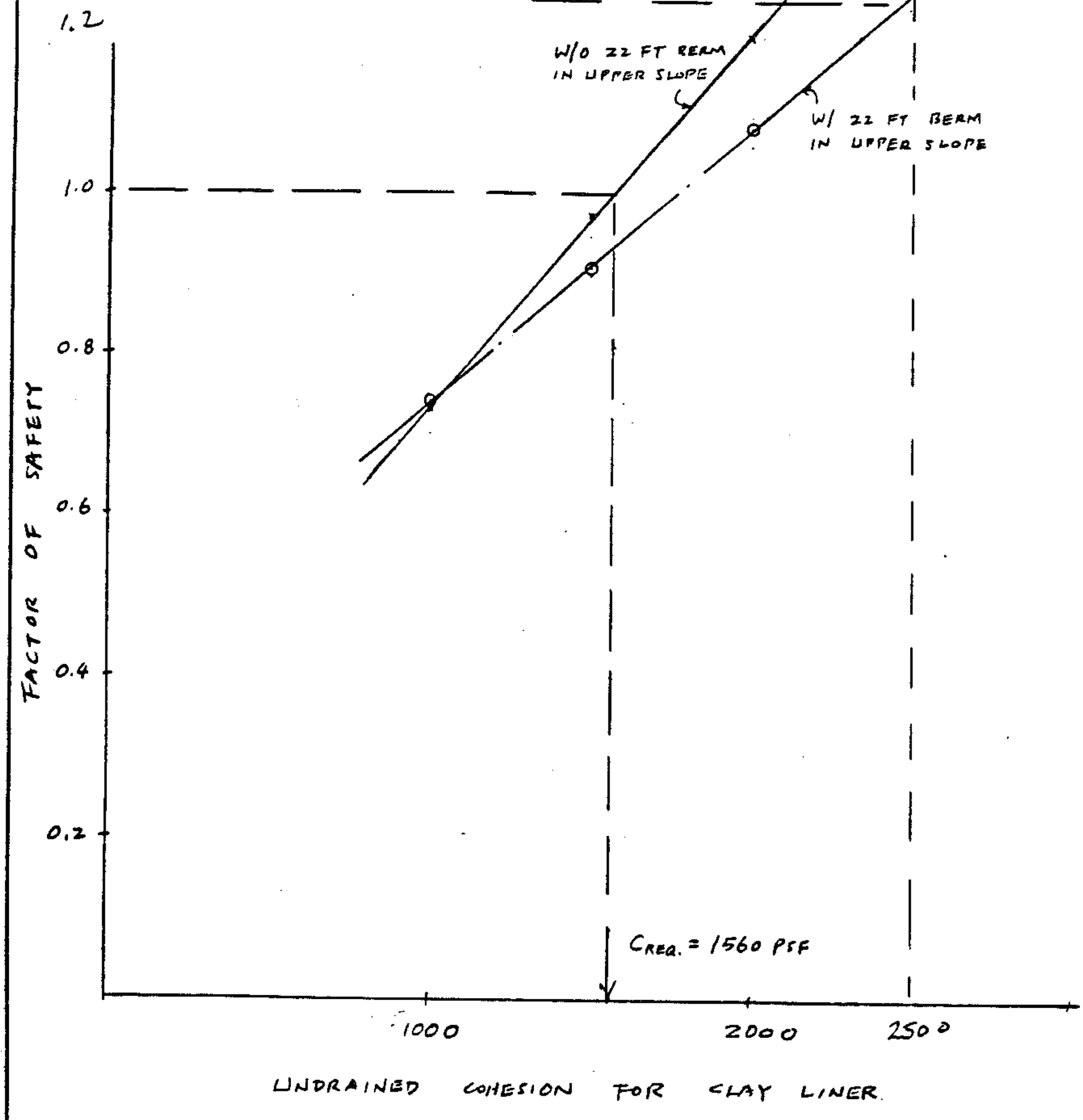
JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO. 13
SUBJECT SUMMARY OF SLOPE STABILITY ANALYSES USING "PC-SLOPE" PROGRAM BY BS DATE 2/19/88
CHK. BY CJM DATE 3-88



NOTE: MORGENSTERN - PRICE METHOD OF ANALYSIS IS USED
ANCHORING EFFECT OF FML IS NOT INCLUDED
EFFECT OF PORE WATER PRESSURE BELOW FML IS INCLUDED
FAILURE SURFACE AT CONTACT BETWEEN FML & UNDERLYING NATIVE CLAY

FIGURE 29

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JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO. ____
SUBJECT LEACHATE COLLECTION SYSTEM BY BS DATE 6-23-88
CHK. BY LY DATE 6/24/88

DETERMINE MAXIMUM PIPE SPACING

A. BASED ON MAXIMUM ALLOWABLE HEAD (REF: SW-869, 1980)

$$L = \frac{2\, h_{max}}{\sqrt{c}\left[\frac{\tan^2\alpha}{c} + 1 - \frac{\tan\alpha}{c}\sqrt{\tan^2\alpha + c}\right]}$$

WHERE h_{max} = ALLOWABLE MAX. HEAD ON LINER = 0.5 FT

$C = \dfrac{q_{DESIGN}}{k_{SAND}}$ ← INFILTRATION, FROM WATER BALANCE CALCULATIONS (FIG. 31)

k_{SAND} ← SAND PERMEABILITY, FOR CLASS II SAND, $\simeq 1\times10^{-2}$ cm/sec

$$= \frac{2.3\times10^{-6}\ \text{cm/sec}}{1\times10^{-2}\ \text{cm/sec}}$$

$$= 2.3\times10^{-4}$$

$\tan\alpha$ = EFFECTIVE SLOPE OF THE BASE OF THE CELL

= 0.0224

$\Rightarrow \tan^2\alpha = 0.000502$

$$\therefore\ L = \frac{2\,(0.5)}{\sqrt{2.3\times10^{-4}}\left[\frac{5.02\times10^{-4}}{2.3\times10^{-4}} + 1 - \frac{0.0224}{2.3\times10^{-4}}\sqrt{5.02\times10^{-4} + 2.3\times10^{-4}}\right]}$$

= 120 FT, SAY 110 FT

FIGURE 30

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JOB Allen Park Clay Mine PROJECT NO. 863470W SHEET NO. 10
SUBJECT Liner Evaluation BY CJM DATE 5-2-87
CHK. BY YBP DATE 5/5/87

Water Balance-

Assume 40% of precipitation evaporates each year (Fenn, et al, 1975) because of bare soil - no vegetation to assist evapotranspiration

Month	J	F	M	A	M	J	J	A	S	O	N	D	Total	
P	5.3	5.4	6.2	6.8	8.5	8.4	7.9	7.1	6.8	6.3	5.9	5.7	80.3	cm
Cslo	.22	.22	.22	.20	.18	.16	.16	.18	.19	.20	.21	.22		
Slo	1.2	1.2	1.4	1.4	1.5	1.3	1.3	1.3	1.3	1.3	1.2	1.3	15.7	cm
Evap	2.1	2.2	2.5	2.7	3.4	3.4	3.2	2.8	2.7	2.5	2.4	2.3	32.2	cm
Perc	2.0	2.0	2.3	2.7	3.6	3.7	3.4	3.0	2.8	2.5	2.3	2.1	32.4	cm

Cell II potentially will be subject to greatest flow prior to placement of final cover. The worst case will occur when filling is essentially complete and relatively flat final surface slopes are formed.

The water balance shown above estimates that the maximum monthly rate of percolation will occur in June. This is the percolation rate on which the leachate collection system design is based.

$$\text{Max perc. rate} = 3.7\text{ cm} / 28\text{ days} = 0.132\ \frac{\text{cm}}{\text{day}}$$

$$\text{Use F.S} = 1.5 \rightarrow q_{design} = \left(0.132\ \frac{\text{cm}}{\text{day}}\right)(1.5) = 0.198\ \frac{\text{cm}}{\text{day}}$$

$$q_{design} = 0.198\ \frac{\text{cm}}{\text{day}} \times \frac{1\text{ day}}{24\text{ hr}} \times \frac{1\text{ hr}}{3600\text{ sec}} = 2.3\times10^{-6}\ \frac{\text{cm}}{\text{s}}$$

$$q_{design} = 2.3\times10^{-6}\ \frac{\text{cm}}{\text{s}}$$

FIGURE 31

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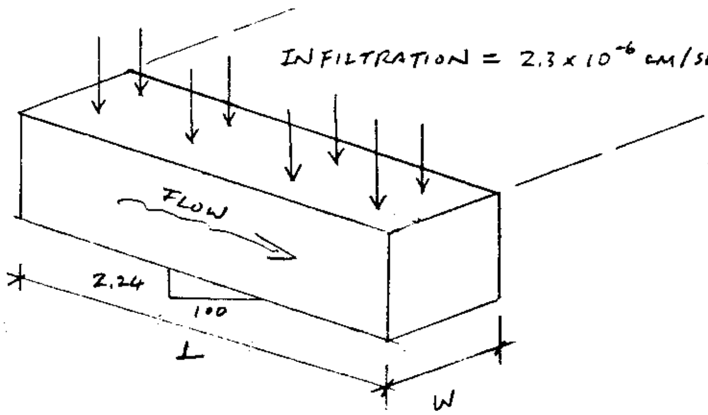
JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO.
SUBJECT LEACHATE COLLECTION SYSTEM BY TBS DATE 6-23-88
CHK. BY LK DATE 6/24/88

B. CHECK TRANSMISSIVITY OF GRANULAR BLANKET ALONG CELL BASE



Q_W = FLOW PER UNIT WIDTH = INFILTRATION × L

$= 2.3 \times 10^{-6}$ CM/SEC × 110 FT × $\frac{12 \text{ IN}}{1 \text{ FT}} \times \frac{2.54 \text{ CM}}{1 \text{ IN}}$

$= 0.0077$ CM²/SEC

TRANSMISSIVITY $T = \frac{Q_W}{i} = \frac{0.0077}{0.0224}$

$= 0.34$ CM²/SEC. ← REQUIRED

CHECK AVAILABLE TRANSMISSIVITY:

$T_{AVAIL.} = k\, t$

$= 1 \times 10^{-2} \frac{\text{CM}}{\text{SEC}} \times 0.5 \text{ FT} \times 12 \frac{\text{IN}}{\text{FT}} \times 2.54 \frac{\text{CM}}{\text{IN}}$

$= 0.15$ CM²/SEC $< T_{REQ'D}$

∴ TRANSMISSIVITY OF SAND GOVERNS.

FIGURE 32

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SUBJECT LEACHATE COLLECTION SYSTEM BY RS DATE 6-23-88
CHK. BY LK DATE 6/24/88

C. DETERMINE MINIMUM PIPE SPACING BASED ON TRANSMISSIVITY

$$\text{INFILTRATION} \times L = T_{AVAIL.} \times i = Q$$

$$\therefore \quad 2.3 \times 10^{-6} \ \frac{CM}{SEC} \times L = 0.15 \ \frac{CM^2}{SEC} \times 0.0224$$

$$L = 1461 \ cm$$

$$= \underline{\underline{48 \ FT}} \leftarrow \text{GOVERN}$$

FIGURE 33

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JOB Allen Park Clay Mine PROJECT NO. 863470W SHEET NO. 1
SUBJECT Liner Evaluation BY GJM DATE 5-2-87
CHK. BY VBP DATE 5/6/87

Minimum Allowable Transmissivity of Leachate Collection System

The drainage net and drainage geotextile will compress under the weight of waste in the cell thus reducing the permeability + carrying capacity of the drainage system. A minimum transmissivity must be maintained to reduce a buildup of head from infiltration.

∴ Calculate the required transmissivity for an infiltration rate of 2.3×10^{-6} cm/s.

Transmissivity ≡ K × t

Where K = permeability, cm/s
t = thickness of drainage layer (cm)

Flow per unit width, $Q/x = Ti$

Where i = gradient

Gradient flow is not to exceed sideslope of 1V : 2H

$$\therefore\ i = 1/(1^2+2^2)^{1/2} = 0.447$$

Flow per unit width is the infiltration rate times the length of slope Q × L L =

$$\therefore\ \frac{Q}{x} = 2.3\times10^{-6}\ \frac{cm}{s} \times 39' \times \frac{12\ in}{ft} \times \frac{2.54\ cm}{in}$$

$$= 2.8\times10^{-3}\ \frac{cm^2}{s}$$

$$\text{Minimum Allowable } T = \frac{Q/x}{i} = \frac{2.8\times10^{-3}}{0.447}\ cm^2/s$$

$$= 6.3\times10^{-3}\ \frac{cm^2}{s}$$

FIGURE 34

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JOB Allen Park Clay Mine PROJECT NO. 863470W SHEET NO. 2
SUBJECT Leachate Collection System Analysis BY CJM DATE 5-2-87
CHK. BY VBP DATE 5/6/87

Determine the normal pressure that the drainage system will be subjected to:

Max pressure at base of slope = $\gamma_{waste}(z) + \gamma_{clay\ cap}(z) + \gamma_{drain}(z)$

$= 75\ pcf(65') + 135\ pcf(5') + 120\ pcf(1')$

$= 5670\ psf$

According to manufacturer's information for PN 4000 (assumed brand although another may be selected) at 7000 psf and $i = 0.5$, $T = 1.5 \times 10^{-3}\ m^2/s$

$T = 1.5 \times 10^{-3} \frac{m^2}{s} = 15 \frac{cm^2}{s}$

$15 \frac{cm^2}{s}$ @ 7000 psf $>> 6.3 \times 10^{-3} \frac{cm^2}{s}$ @ 5670 psf

$\therefore$ OK

FIGURE 35

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JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO.
SUBJECT LEACHATE COLLECTION SYSTEM BY TS DATE 6-23-88
CHK. BY LK DATE 6/24/88

PIPE DEFLECTION CALCULATIONS

6" ϕ HDPE SDR 7.3 PIPES MANUFACTURED BY PLEXCO HAS BEEN SPECIFIED FOR USE.

DEFLECTION WILL BE CALCULATED BASED ON A FORMULA DEVELOPED BY SPANGLER (REFERENCE: ASCE MANUAL OF PRACTICE, NO. 37, CHP. 9, SEC. E, SUBSEC. 1, 1970 ED.)

$$\Delta y = D_e \frac{K W r^3}{EI + 0.061 E' r^3}$$

WHERE Δy = HORIZ & VERT. DEFLECTION OF THE PIPE (INCH)

D_e = FACTOR COMPENSATING FOR LAG TIME, USUALLY TAKEN AS 1.5

r = MEAN RADIUS

E = MODULUS OF ELASTICITY OF PIPE MATERIALS

E' = MODULUS OF PASSIVE SOIL RESISTANCE.

K = BEDDING CONSTANT, CONSERVATIVELY TAKEN AS 0.1

I = MOMENT OF INERTIA OF PIPE

$= t^3/12$

W = VERT. LOAD ACTING ON PIPE PER UNIT LENGTH

$= \sigma_v B_c$

FIGURE 36

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JOB ALLEN PARK CLAY MINE PROJECT NO. 863470W SHEET NO.
SUBJECT LEACHATE COLLECTION SYSTEM BY BS DATE 6-23-88
CHK. BY LV DATE 6/24/88

DETERMINE W:

VERTICAL PRESSURES ON PIPE, σ_v

$$\gamma_{WASTE} = 75 \text{ pcf} \times 60 \text{ FT} = 4500$$
$$\gamma_{CAP} = 135 \text{ pcf} \times 7 \text{ FT} = 945$$
$$\gamma_{LINER} = 135 \text{ pcf} \times 5 \text{ FT} = 675$$
$$\gamma_{DRAIN} = 120 \text{ pcf} \times 1 \text{ FT} = 120$$
$$\Sigma\sigma_v = 6240 \text{ PSF}$$

TO ACCOUNT FOR PERFORATIONS IN PIPE,

$$l_p = \tfrac{1}{4}'' / \text{HOLE} \times 6 \text{ HOLES/FT} = 1.5 \text{ IN/FT}$$

$$(\sigma_v)_{DESIGN} = \frac{12}{12 - l_p} \times (\sigma_v)_{ACTUAL}$$

$$= \frac{12}{12 - 1.5} \times 6240$$

$$= 7131 \text{ PSF}$$

FROM ASTM F-714, TABLE 4,

B_c = 6.625 IN.
← OUTSIDE DIA. OF PIPE

$$\therefore W = B_c \sigma_v$$

$$= 6.625 \text{ IN} \times 7131 \frac{LB}{FT^2} \times \frac{1}{144} \frac{FT^2}{IN^2}$$

$$= 328.1 \text{ LB/IN}$$

FIGURE 37

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JOB ALLEN PARK CLAY MINE PROJECT NO. 86347OW SHEET NO.
SUBJECT LEACHATE COLLECTION SYSTEM BY BS DATE 6-23-88
CHK. BY LK DATE 6/24/88

ALSO, $r = \frac{B_c}{2} = 3.31$ IN.

$E^* = 133{,}000$ PSI FOR PE 3408

$E' = 2250$ PSI FOR PEA GRAVEL

$I = t^3/12 = (0.908)^3/12 = 0.0624$ IN^3

$$\therefore \quad \Delta y = \frac{(1.5)(0.10)(328.1)(3.31)^3}{133000(0.0624) + 0.061(2250)(3.31)^3}$$

$$= 0.134 \text{ IN.}$$

$$\frac{\Delta y}{B_c} = \frac{0.134}{6.625} = 0.020 = 2.0\%$$

ACCORDING TO PIPE MANUFACTURER, DEFLECTION OF PIPE SHOULD NOT EXCEED 2% ∴ OK

FIGURE 38

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SUBJECT BY DATE 6-24-88
CHK. BY DATE

FIG. 39 IS DELETED

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JOB Allen Park Clay Mine PROJECT NO. 863470W SHEET NO. 5
SUBJECT Liner Engineering Report BY GTM DATE 4-25-88
CHK. BY VBP DATE 4-29-88

LEACHATE COLLECTION SYSTEM

Assessment of maximum load 6-inch HDPE SDR 7.3 pipe can withstand

The purpose of this calculation is to determine the maximum equipment load that will not result in excess deflection of the HDPE pipe.

rearranging the deflection equation:

$$W = \frac{\Delta y\,(EI + 0.061\,E' r^3)}{D_e K r^3}$$

$$\Delta y = \frac{(\text{max. allow. deflection})B_c}{100} = \frac{(2.0)(6.625")}{100} = 0.1325$$

$$= \frac{(0.1325)\left[(110{,}000\,psi)(0.0624\,in^3) + 0.061(2000\,psi)(3.31)^3\right]}{(1.5)(0.10)(3.31\,in)^3}$$

$W = 275$ lb/in

$$\sigma_T(\text{design}) = 275\,\frac{lb}{in} \times \left(\frac{1}{6.625"}\right) = 41.5\ psi$$

∴ Maximum equipment load should not exceed 41.5 psi in post closure stages of landfill

FIGURE 40

Pipe Capacity + Minimum design Slope

Infiltration (worst case) = 23×10^{-6} cm/s

= 7.55×10^{-8} ft/s

Area of Proposed Cell

Cell IIA = 95.01 in² × $\left(\frac{40\text{ ft}}{\text{in}}\right)^2$ = 152,016 ft²

Cell IIA = 49.98 in² × $\left(\frac{40\text{ ft}^2}{\text{in}^2}\right)$ = 79,968 ft²
≈ 34% of total Area

Cell IIB is the largest area drained by one manhole

Qdesign = $(7.55 \times 10^{-8}$ ft/s$)(79,968$ ft²$)$

= 6.0×10^{-3} ft³/sec

Check this flow in 4" ϕ pipe flowing full - use Manning equation

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$ $\quad Q = VA$

$V = \frac{6.0 \times 10^{-3}\text{ ft}^3/\text{s}}{\pi/4\ (1/3\text{ ft})^2} =$

= 0.0692 ft/sec

$R_h = D/4 = 4''/4 = 1'' = 0.083$ ft
(hydraulic radius flowing full)

use n = 0.015 (conservative for PE pipe)

$0.0692 \frac{\text{ft}}{\text{sec}} = \frac{1.49}{0.015} (.083)^{2/3} S^{1/2} = 18.9 S^{1/2}$

S = 0.13% (for single collector main)

Minimum Design Slope = 1% → OK

JOB Allen Park Clay Mine PROJECT NO. 863470W SHEET NO. 3
SUBJECT Leachate Collection System Analysis BY CJM DATE 3-17-87
CHK. BY TBS DATE 3/23/87

Open Area on Perforated Pipe

Pipe capacity $q_{design} = 2.3 \times 10^{-6}$ cm/s

Maximum distance travelled = 103'

Assume 1 ft wide strip

$$Q = 103\,ft \times 1\,ft \times 2.3 \times 10^{-6} \frac{cm}{s} \times \frac{1\,inch}{2.54\,cm} \times \frac{1\,ft}{12\,in}$$

$$Q = 7.77 \times 10^{-6}\ ft^3/s$$

limit entrance velocity to 0.1 ft/s (Groundwater + Wells, Johnson Division, 1975)

Calculate Entrance Velocity:

6 holes per foot of pipe

holes = 1/4" ϕ

$Q = VA$

$$V = \frac{7.77 \times 10^{-6}\ ft^3/s}{6 \times \pi \left(\frac{(.25/12)^2}{4} \right)} = 3.8 \times 10^{-3}\ \frac{ft}{s}$$

Calculated Entrance Velocity should be less than $0.1 \frac{ft}{s}$

$3.8 \times 10^{-3} \frac{ft}{s} \ll 0.1 \frac{ft}{s}$, $\therefore$ OK, ie hole diameter is adequate

Examine Pipe

Pipe Perforations are 1/4" ϕ

For Circular holes

$$\frac{\text{Min filter } D_{85}}{\text{Hole diameter}} \geq 1.2 \quad \text{(USEPA - SW 870)}$$

the required drainage material is to be 3/8" peastone

Minimum D_{85} of filter material $= 1.2(0.25) = 0.3$ in

D_{85} of peastone $= 0.375$ in

$0.375 \text{ in} > 0.3 \text{ in}$, $\therefore$ OK,
peastone should not clog pipe

Permeability Criterion of the Drainage fabric - Assume Propex 4545 although another may be selected

Propex 4545 is to be placed between the clean sand, $K = 1 \times 10^{-2}$ cm/s and the 3/8" peastone material around the pipes

$K_{sand} = 1 \times 10^{-2}$ cm/s

Giroud (1981) suggests: $\frac{K_f}{K_s} = 10$

According to manufacturers information,

$(K_f)_{min} = 2 \times 10^{-2}$ cm/s

$(K_f)_{max} = 3 \times 10^{-1}$ cm/s

$0.02 < K_f < 0.3$

$K_f = 10 K_s = 10(1 \times 10^{-2}) = 0.1$ cm/s

$\therefore$ A filter fabric with a permeability of 0.1 cm/s must be selected

D-6f Run-On Control Systems-Michigan Act 64 of Public Acts 1979 and 40 CFR 270.21 (b) (2)

The Administrative Rules state that to minimize leachate generation:

"All disposal facilities shall have diversion structures capable of diverting all surface runoff water away from the active portions of the disposal facility for a 24 hour, 100 year storm."

D-6f (1) Calculation of Peak Flow

1. **Description of Hydrologic method used to estimate peak flow rates.**

2. **Data and Input parameters.**

   - **Soil classification determined from available soil boring data.**
   - **Runoff areas and slopes determined from available topographic maps.**
   - **Type of ground cover determined from field observation.**
   - **On-site drainage data taken from present landfill plans.**

3. Determine Peak Flow Rate- 100 year, 24 hour storm

Using U.S. Weather Bureau Technical Paper No. 40- 100 year, 24 hour rainfall is 4.6 inches for this area.
Using U.S. Soil Conservation Service TR-55 Graphical Method

Ditch Along Relocated Haul Road-SW of Cell II (Area A)

-SCS Method TR-55 (Figs. 5, 7, & 9)
-Type "B" Soil
-Slopes-Moderate, 5% average
-Most offsite area would be classified as "cultivated"
-From the above a runoff curve number of 80 would be conservative.
-Drainage area tributary to the most downstream ditch point of this section; midway along the west side of Cell II= 4.56 acres.
-Runoff Depth= 2.55 inches
-Peak discharge = 14 cfs

(See Hydrological Analysis Charts in Appendix)

Ditch Along Relocated Haul Road-NW of Cell II (Area B)

-SCS Method TR-55
-Type "B" Soil
-CN of 80
-Most offsite area would be classified as "cultivated"
-Runoff Depth = 2.55 inches

The above parameters apply to the entire site unless otherwise noted.

-Slopes 5% average
-Drainage area tributary to the downstream ditch point at the northwest corner of Cell II = 7.41 acres.
-Peak Discharge = 21 cfs

<u>Ditch Along Hazardous Waste Haul Road-NE of Cell II</u> (Area D)

-Slopes 20% average (for Cell II interim slopes)
-Drainage area tributary to the most downstream ditch point at the midpoint of the ditch run = 1.02 acres
-Peak Discharge = 4 cfs

<u>Ditch Along Hazardous Waste Haul Road-NW of Cell II</u> (Area C)

-Slopes 20% average (for Cell II interim slopes)
-Drainage area tributary to the most downstream ditch point at the west end of the Hazardous Waste Haul Road = 2.00 acres
-Peak Discharge = 8 cfs

<u>Ditch Southwest of Cell II Along Existing Haul Road</u> (Area E)

-Slopes 4% average
-Drainage area tributary to the southwest corner of Cell II where the perimeter ditch turns easterly = 0.75 acres
-Peak Discharge = 3 cfs

<u>Ditch Along Southwest Part of Cell II</u> (Area F)

-Slopes 4% average
-Drainage area tributary to the southerly midpoint of Cell II where the ditch jogs to the north = 5.40 acres
-Peak Discharge = 16 cfs

<u>Ditch Along the Southeasterly Part of Cell II</u> (Area G)

-Slopes 6% average
-Drainage area tributary to the southeast corner of Cell II = 7.84 acres
-Peak Discharge = 21 cfs

<u>Ditch Along the South Side of Existing Cell I</u> (Area H)

-Slopes 6% average
-Drainage area tributary to the Sedimentation Pond = 10.26 acres
-Peak Discharge = 25 cfs

D-6f (2) <u>Design and Performance</u>

The basic Run-On control facility is the drainage ditches that run along the north, west and south sides of Cell II. As the cell is prepared for operation, a clay dyke is constructed around the cell, through the upper sand formation and extending

up to 10 feet above adjacent ground surfaces. This dike will prevent run-on from entering Cell II from areas outside the cell.

The drainage ditches will be checked for adequate size and capacity.

Ditch Design Calculations

Formulas;

$$A = \frac{Q}{V} \quad \text{(Continuity)}$$

$$Q = \frac{1.486}{N} A \, (R^{2/3}) \, S^{1/2} \quad \text{(Manning Equation)}$$

Section A

Qd = 14 cfs
Sb = 0.60%
Ss = 2H:1V
N = 0.03

Depth = 1.55 '

$$Q = \frac{1.486}{.03} \times 4.81 \times (.69^{2/3}) \times (.006^{1/2})$$

$Q = 14.4$ cfs $\simeq$ 14 cfs

V = 14 / 4.81 = 2.9 fps (OK)

Section B

Qd = 21 cfs
Sb = 0.65 %
Ss = 2H:1V
N = 0.03

Depth = 1.8 '

$$Q = \frac{1.486}{.03} \times 6.48 \times (.805^{2/3}) \times (.0065^{1/2})$$

$Q = 22.4$ cfs $\simeq$ 21 cfs

V = 21 / 6.48 = 3.2 fps (OK)

Section D

Qd = 4 cfs
Sb = 0.70%
Ss = 2H:1V
N = 0.03

Depth = 0.95 '

$$Q = \frac{1.486}{.03} \times 1.81 \times (.426^{2/3}) \times (.007^{1/2})$$

$Q = 4.2$ cfs $\approx$ 4 cfs

V = 4 / 1.81 = 2.2 fps (OK)

Section C

Qd = 8 cfs
Sb = 1.00%
Ss = 2H:1V
N = 0.03

Depth = 1.15 '

$$Q = \frac{1.486}{.03} \times 2.64 \times (.514^{2/3}) \times (.01^{1/2})$$

$Q = 8.4$ cfs $\approx$ 8 cfs

V = 8 / 2.64 = 3.0 fps (OK)

Section E

Section E consists of an existing ditch adequate for continued use as no change in it's upstream area is proposed.

Section F

Qd = 16 cfs	Depth = 1.7 '
Sb = 0.50%	$Q = \frac{1.486}{.03} \times 5.78 \times (.761^{2/3}) \times (.005^{1/2})$
Ss = 2H:1V	Q = 16.8 cfs ≈ 16 cfs
N = 0.03	
	V = 16 / 5.78 = 2.77 fps (OK)

Section G

Qd = 21 cfs	Depth = 1.9 '
Sb = 0.50%	$Q = \frac{1.486}{.03} \times 7.22 \times (.849^{2/3}) \times (.005^{1/2})$
Ss = 2H:1V	Q = 22.6 cfs ≈ 21 cfs
N = 0.03	
	V = 21 / 7.22 = 2.9 fps (OK)

Section H

Qd = 25 cfs	Depth = 2.0 '
Sb = 0.50%	$Q = \frac{1.486}{.03} \times 8 \times (.895^{2/3}) \times (.005^{1/2})$
Ss = 2H:1V	Q = 26.0 cfs ≈ 25 cfs
N = 0.03	
	V = 25 / 8 = 3.1 fps (OK)

Since ditch bottoms will be a minimum of 2.5 feet below adjacent road surfaces and a minimum of 3.5 feet below the top of adjacent dikes, there will be no problem handling run-on and no problem of drainage overtopping the dike causing inflow into the active area.

Minimum Freeboard = 3.5 ft. - 2.0 ft. = 1.5 feet.

Structural Design

All run-on control conveyances are in the form of open ditches constructed of CL or CH classification clay soil material.

Ditch velocities for the various design conditions are very low and non-erosive. Generous freeboard depths are also available in all ditches.

D-6f (3) Construction

The first step in the ditch construction procedure is the excavation of the sand formation and construction of the clay cut off walls extending from the existing clay formation up to plan grades. This cut off wall forms one side of the ultimate ditch section, and provides an engineered clay barrier between the ditch and Cell II, preventing both surface overflow and underground intrusion into the cell.

The ditch along the west side of Cell II, adjacent to the existing haul road is already existing, but will be relocated per plan when perimeter dike construction and haul road relocation occurs.

These ditches protect the north, south, and west sides of the Cell from run-on from adjacent land areas. To the east, grades on Cell I provide surface run-off away from Cell II to the existing sediment pond.

Ditch capacities and freeboard conditions have been set forth previously.

D-6f (4) Maintenance

The generally low velocities and rates of flow, combined with substantial excess available capacity substantially reduce any expected need for repair, and also reduce the possibility that a failure could result in run-on entering the active areas.

However, all ditches will be inspected on a regular basis, not exceeding 3 month intervals, and any potential failure areas repaired as appropriate. Active landfill areas that are still below ground will be inspected on a weekly basis or after each storm event to see that no failures in the drainage system have occurred.

D-6g Run-Off Control System-Michigan Act 64 of Public Acts 1979 and 40 CFR 270.21(b)(3)

The Administrative Rules state that to minimize hazards from run-off of contaminated liquid:

"Surface water run-off, up to the quantity anticipated from a 24 hour, 100 year storm, from the active portions of a disposal facility shall be collected and confined to a point source..."

D-6g(1) Calculation of Peak Flow

We are concerned here about the portion of the landfill that is in a technically "active" stage at any one time. This is understood to mean all areas where waste has been placed, that have not yet received completed final cover.

The general design concept to be used, will consist of containing as much of the direct active area run-off as possible, in or on the cell, while minimizing the amount of this runoff that is allowed to enter the stored waste. After collection and containment, and after the storm event is over, the runoff will be removed for physical evaluation, and possible treatment prior to discharge. Method of discharge will be determined by the end quality of the runoff.

Run-off Calculation Assumptions

Formulas are the same as used in the Run-on computation:

- -Type "C" soil
- -Surface character "uncultivated"
- -Runoff Curve (CN) of 85
- -Direct Runoff = 3.0 inches (See Hydrological Analysis Chart-Runoff From Active Landfill Areas, in Appendix)

Active Area Assumptions

Active fill areas contribute runoff which must be contained prior to treatment and/or discharge. The areas of concern are those without final cover since all final covered areas slope away from the active area,&are protected by the final cover membrane liner. Therefore, the critical element is the largest open fill module while that module is in progress, plus the twenty feet of space from the toe of the nearly complete module to the temporary containment berm.

Runoff Calculation per Unit Width of Cell-

Assuming a one foot wide section, perpendicular to the direction of filling, with a maximum length of 220 feet (in module #2) plus a distance of 20 feet to the temporary containment dike, results in a total area of 240 square feet.

Runoff Volume = $240' \times 1' \times \frac{3''}{12''}$ = 60 cubic feet (for largest module)

Available Storage = 20 ' x 4' x 1' = 80 cubic feet
(1' freeboard at dike)

The design storm therefore equals a 3' temporary impoundment against the 5' high temporary containment berm, for the maximum sized module. Most of the other modules have a much smaller width, on the order of 130 feet, including the 20 ft. space to the dike and therefore would have much smaller impoundments.

Typical Runoff Volume = $130' \times 1' \times \frac{3''}{12''}$ = 32.5 cubic feet.

This equates to a typical impoundment depth of 1.6 feet.

Lesser storms than the 100 year frequency event would generate less runoff and smaller impoundments.

D-6g(2) Design and Performance

The remaining unfilled cell bottom area has its runoff provided for through isolation from the portion containing waste by a series of containment dikes in the cell. The runoff in the "clean" portion of the cell is collected at the low end and pumped out to the natural drainage system.

The area of exposed waste will not have any separate, dedicated runoff collection system. Several potential temporary means were considered, but discarded due to practical difficulties in maintaining them during the waste placement operation. What is proposed for this stage of landfill operation is to allow the runoff falling directly on the active work area to runoff & to infiltrate into the waste and be pumped out of either the temporary leachate sumps on the waste side of the bottom cross dykes, or when the bottom is filled, infiltrated water will be pumped out of the permanent leachate collection manhole. It is felt that this is the most practical way of handling infiltration during the landfill operation, and will result in some buffering of leachate volumes to be handled. While this may result in higher contaminant levels in the leachate, it is felt that this material must be handled as leachate anyway. Emphasis will be placed on keeping the exposed waste working face as small as possible, getting above ground as soon as possible, and placing intermediate or final cover as soon as possible. All of these will tend to reduce the quantity of contaminated water. It is recognized that this is the most critical period for control of leachate and it is our opinion that time of exposure reduction via final cover construction is the most effective way to manage leachate generation.

D-6g(3) Construction

The construction elements outlined above are part of the normal required construction of the landfill, and are covered elsewhere in this report, or in the accompanying plans.

D-6g(4) Maintenance

Since most construction elements are part of normal landfill construction, their maintenance is part of that activity as well. Temporary cross berms will be inspected weekly or after each storm event to check for weak or failed areas., which if found will be repaired as soon as weather permits. Temporary surface containment berms will be inspected on the same schedule. Drain pipes will be checked on the same schedule to see that no failure, silting in, or other blockage has occurred.

Runoff will be promptly pumped out after each storm event, tested if necessary and appropriately discharged as previously provided for.

D-6h Management of Collection and Holding Units 40CFR 270.21 (b) (4)

The Facility Standard states that:

> "Collection and holding facilities (e.g., tanks or basins) associated with run-on and run-off control systems must be emptied or otherwise managed expeditiously after storms to maintain design capacity of the system."

The only collection and holding facility proposed, is for containment of clean runoff in the trench bottom beyond the active face temporary berm, and containment of surface runoff until final cover is completed. Both of these facilities will be pumped dry immediately after each storm event. If there is any question of contamination of any of this runoff, it shall be tested to determine if any treatment is required prior to discharge.

Runoff that is allowed to pass through the active area will be removed from the leachate sump or leachate collection manhole for evaluation and possible treatment prior to discharge to the municipal sanitary sewer system. All non-contaminated runoff will be discharged to the natural drainage course. All contaminated runoff will be handled as leachate and processed through the leachate holding tank, with ultimate discharge to Wayne County's public sanitary sewer manhole #23A.

D-6i <u>Control of Wind Dispersal</u> 40CFR 270.21 (b) (5)

The Facility Standard states that:

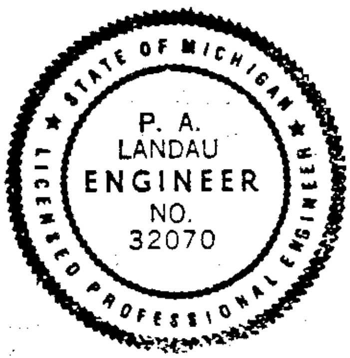
> "If the landfill contains any particulate matter which may be subject to wind dispersal, the owner or operator must cover or otherwise manage the landfill to control wind dispersal."

Particulate emissions caused by wind erosion of landfill wastes or soil cover material can be minimized by various forms of physical, chemical or vegetative stabilization. Wind dispersal of landfilled wastes will be controlled primarily by regular compaction of waste material, and daily application of cover material over all exposed waste surfaces during the active life of the landfill. As individual cells are brought to final grade, the final clay cover and synthetic fabric top liner will be constructed over these areas, thus further sealing the landfill and isolating waste material from potential wind dispersal.

Wind dispersal of daily cover material must also be controlled. This will be done by application of dust settling water spray on a regular basis as necessary. Ford Motor Company presently has a 2500 gal. water tank truck on site on a full time basis. This is presently manned and operated by D & S Liquid Transport under a contract with Ford Motor Co. Ford will maintain this agreement or a similar one on a continuing basis for the life of the landfill. If necessary during extremely dry periods, they will also arrange for chemical stabilization applications through the same firm. Regular water spray applications will also be applied to interior unpaved access roads to control dust and blowing soils. Truck wheel wash installed to minimize track out and resulting fugitive dust.

Final cover and vegetative growth will be used to permanently stabilize the final landfill surface. Upon completion of the final clay and synthetic fabric cover, topsoil and seed, fertilizer and mulch will be applied to establish a final dense grass cover on the landfill.

This is to certify that the above stated provisions of sections D-6f through D-6i are required to comply with the provisions of 40 CFR 270.21 & Michigan Act 64 of Public Acts of 1979.



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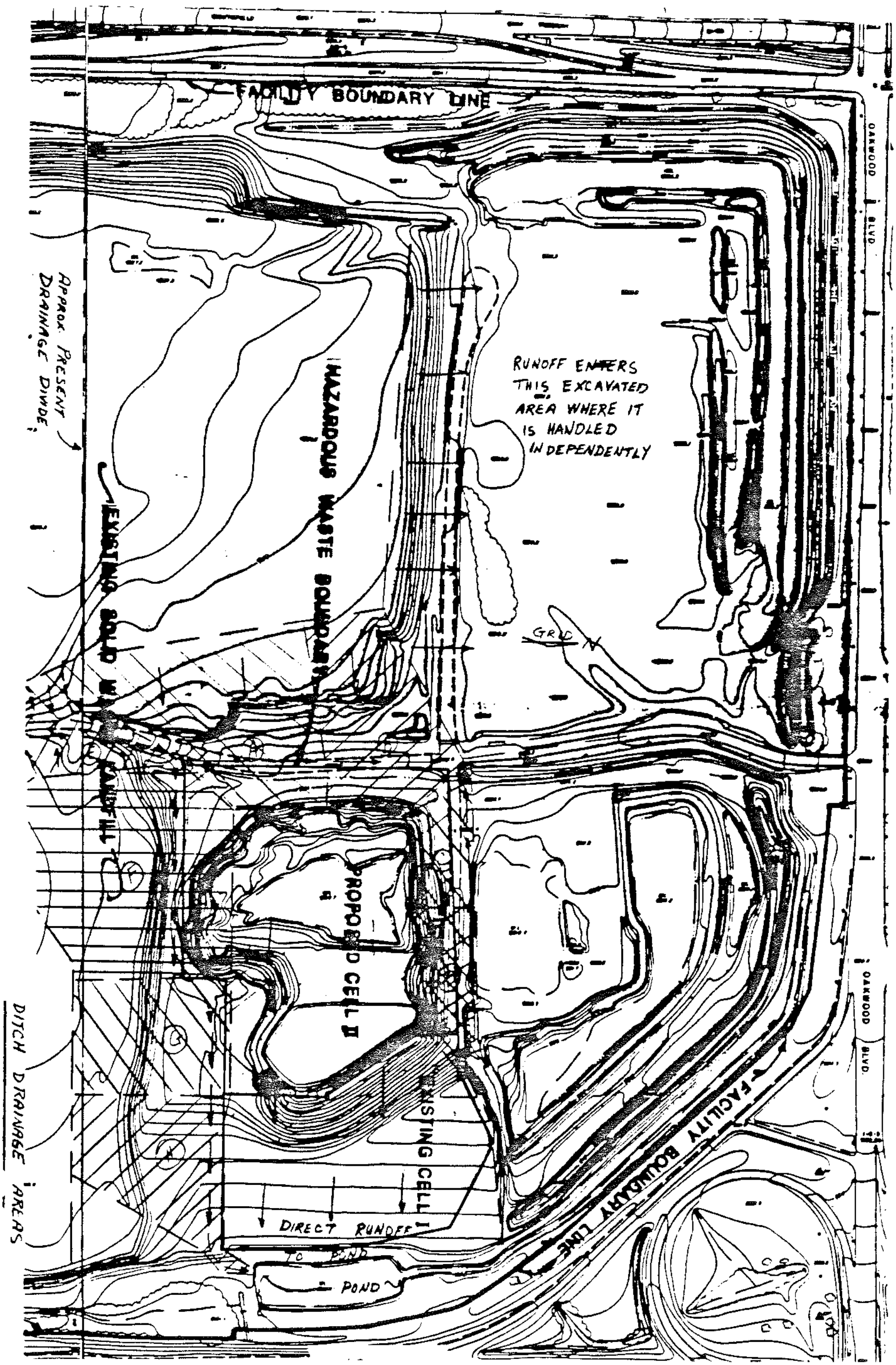
FACILITY BOUNDARY LINE
OAKWOOD BLVD.
RUNOFF ENTERS THIS EXCAVATED AREA WHERE IT IS HANDLED INDEPENDENTLY
APPROX. PRESENT DRAINAGE DIVIDE
HAZARDOUS WASTE BOUNDARY
EXISTING SOLID
GRID N
PROPOSED CELL II
EXISTING CELL I
DIRECT RUNOFF TO POND
POND
FACILITY BOUNDARY LINE
OAKWOOD BLVD.
DITCH DRAINAGE AREAS

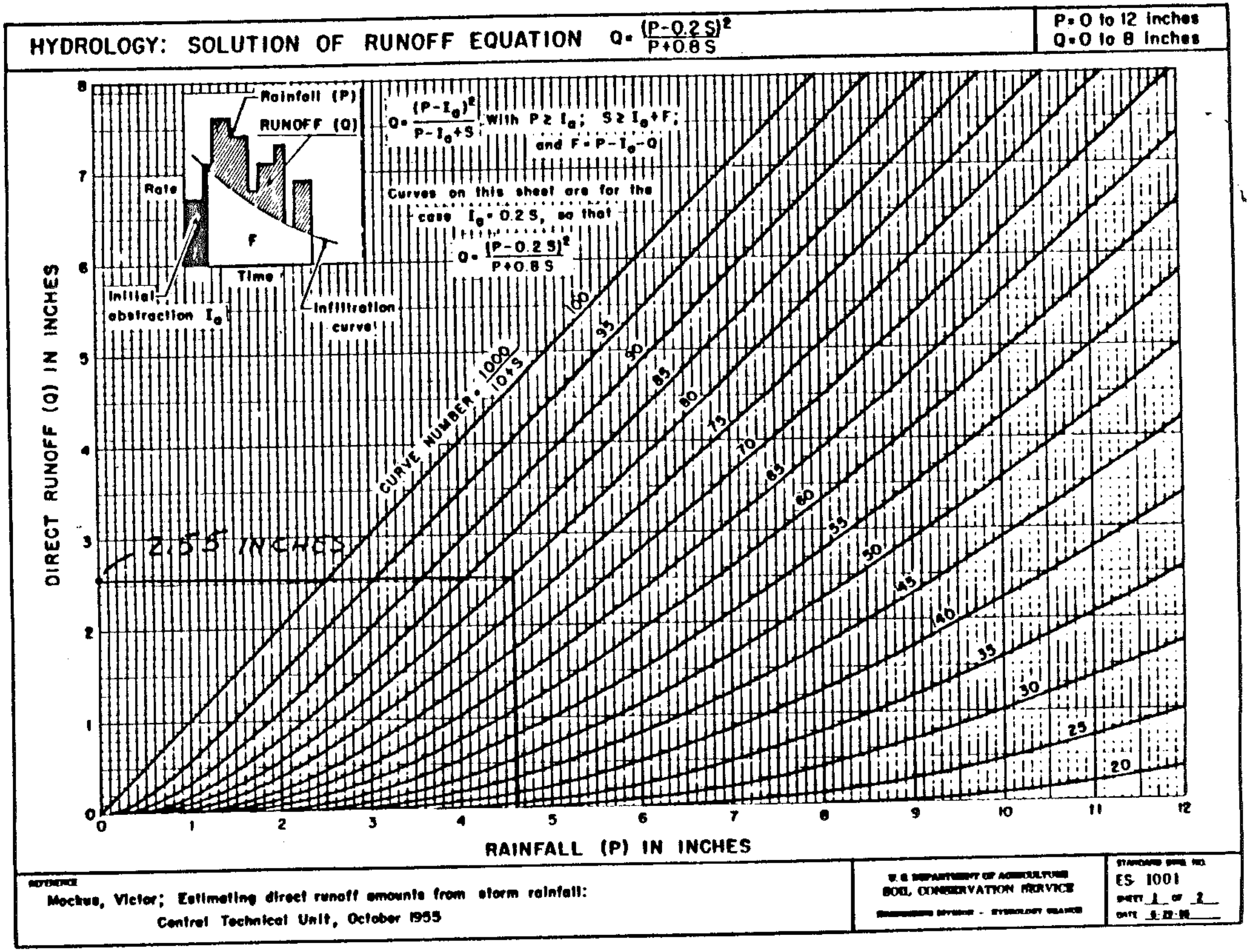

FIGURE 5. Graphical Solution of Rainfall-Runoff Equation

HYDROLOGIC ANALYSIS FOR DITCH DESIGN

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APPENDIX

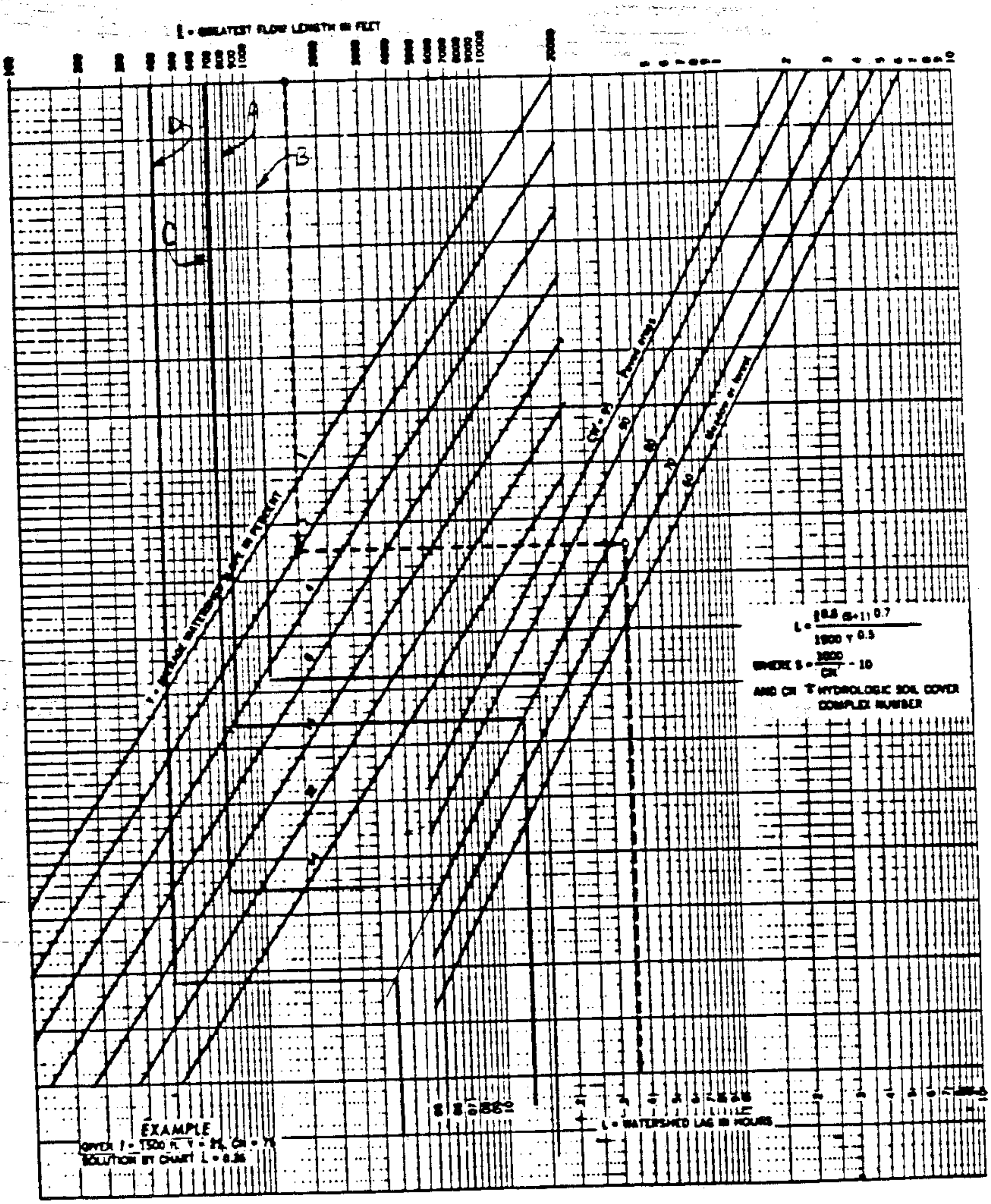

Figure 7.—Curve number method for estimating lag (L)

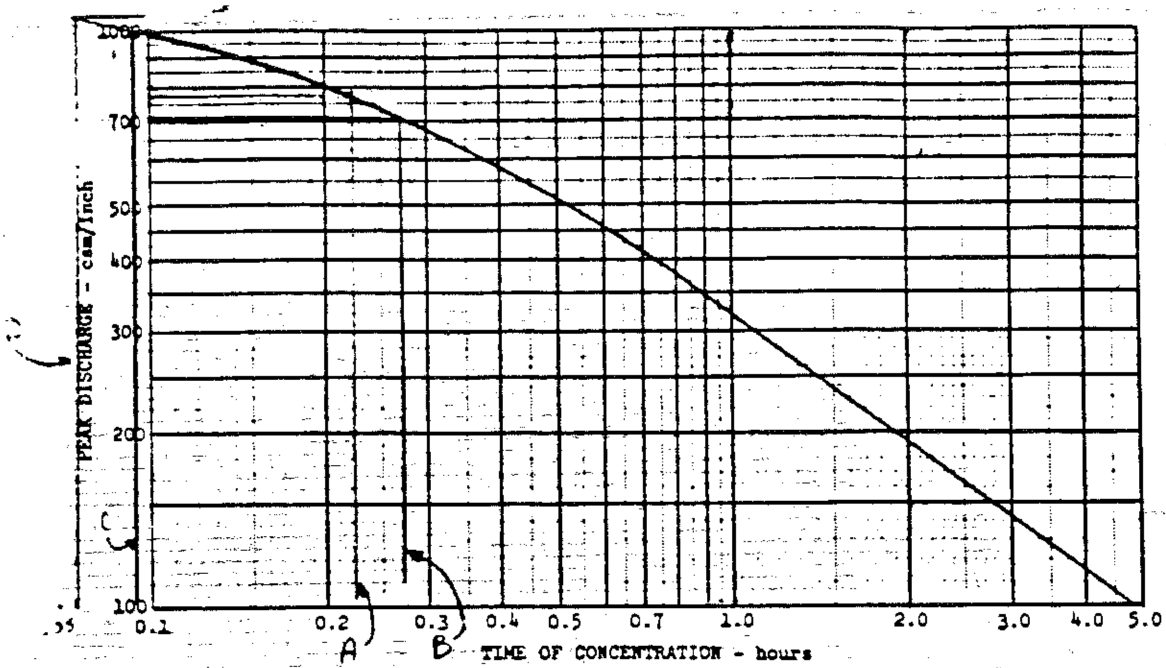

Figure 9. Peak discharge in csm per inch of runoff versus time of concentration (T_c) for 24-hour, type-II storm distribution.

To further define limitations on the graphical method the results of numerous TR-20 runs were compared with estimates of peak discharge made with the graphical method. The runs were made for ranges of the time of concentration (hours), the precipitation volume (inches), and the curve number of 0.5 to 5.0 hours, 1.0 to 10.0 inches, and 50 to 95 curve number units, respectively. The results indicate that the graphical method is a valid approximation of TR-20 as long as the initial abstraction is less than 25 percent of the total 24-hour rainfall; this constraint is easily assessed using the following tabular representation of the constraint, which relates the curve number (CN) and the minimum precipitation:

CN	minimum precipitation
50	8.00 inches
60	5.33
70	3.42
80	2.00
90	0.88
95	0.42

SECTIONS A, B, C, D

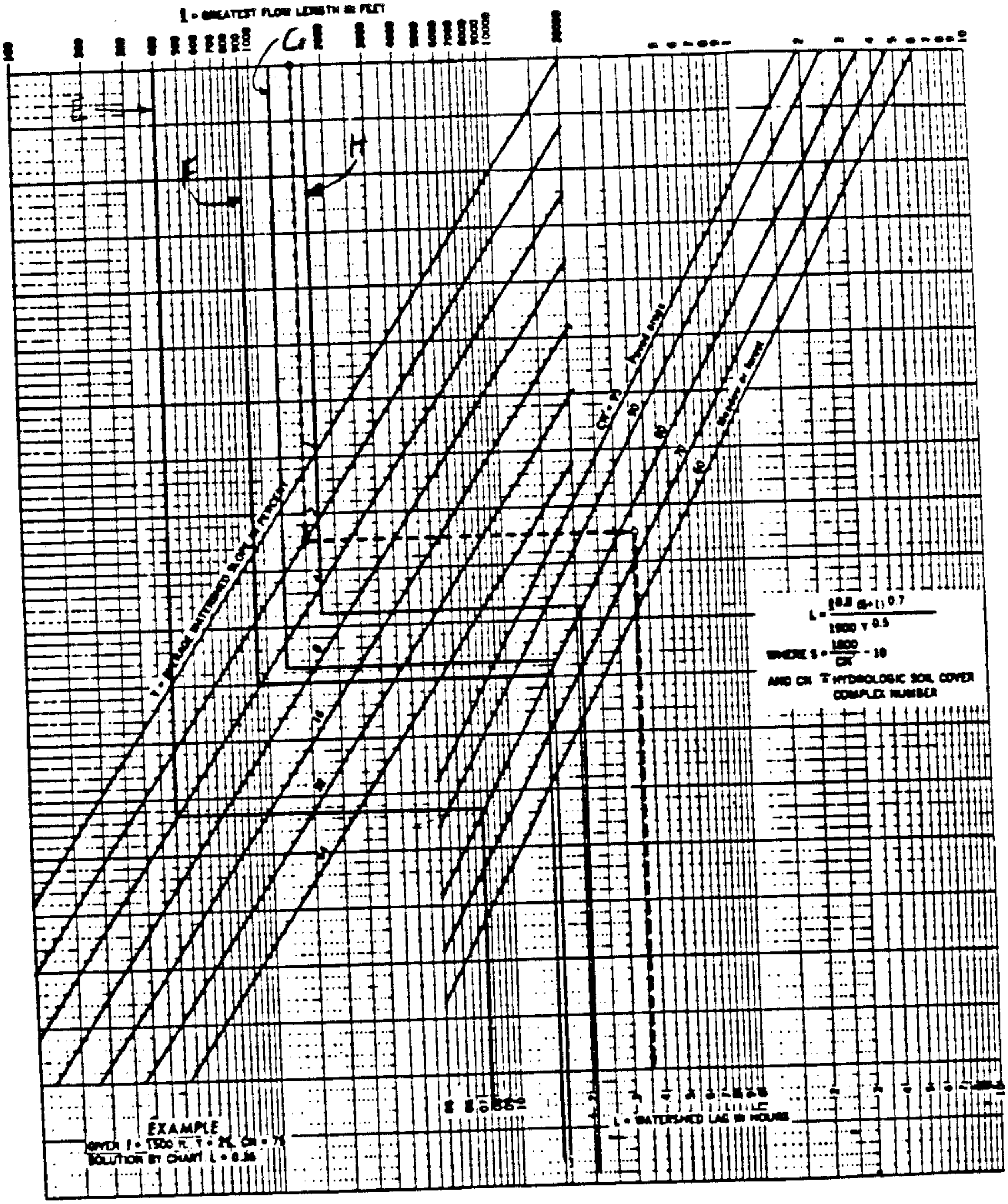

Figure 7.—Curve number method for estimating lag (L)

SECTIONS E, F, G, H

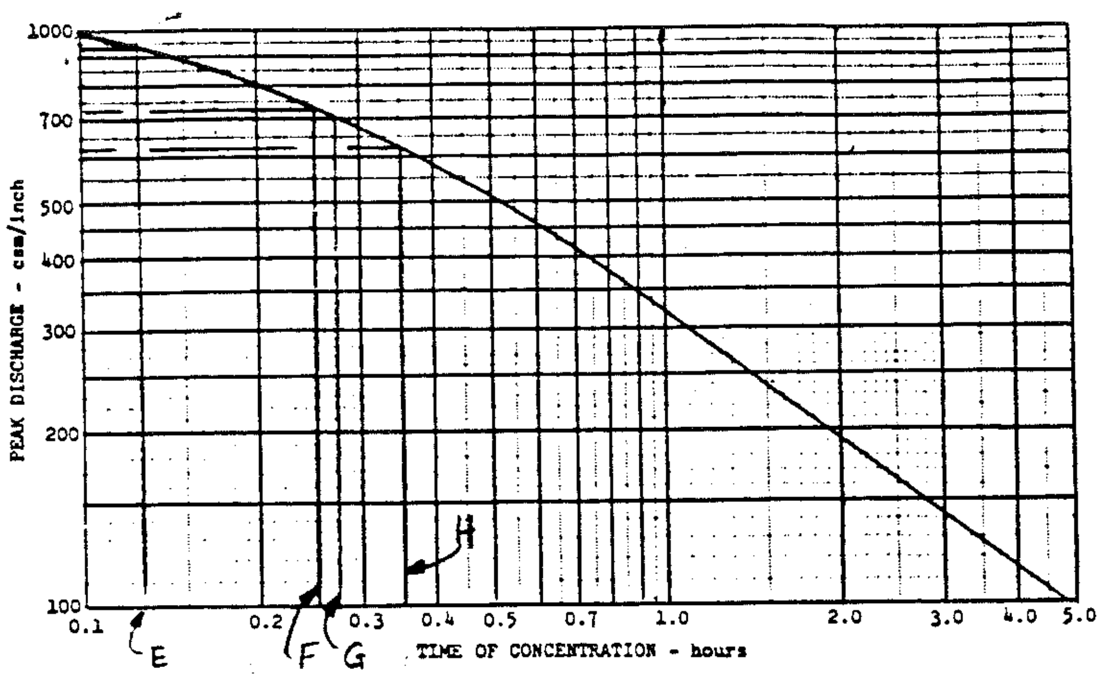

Figure 9. Peak discharge in csm per inch of runoff versus time of concentration (T_c) for 24-hour, type-II storm distribution.

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CN	minimum precipitation
50	8.00 inches
60	5.33
70	3.42
80	2.00
90	0.88
95	0.42

SECTIONS E F G H

TABLE 3. COMPUTATION SHEET: TR-55 GRAPH METHOD - SECTION A

1. Estimate the volume of runoff
   - *a. T = 100 (years): return period for design
   - *b. P = 4.6 (inches): 24-hr, T-year precipitation volume (i.e., depth)
   - *c. CN = 80 : runoff curve number
   - d. Q = 2.55 (inches): runoff volume obtained from Eq. 7 or Fig. 5

2. Drainage Area: A = .0071 (Square miles) (4.56 Ac)

3. Estimate Time-of-Concentration (use either the lag method or the velocity method)

LAG METHOD

- *a. CN = 80
- *b. Slope = 5% (%)
- *c. hydraulic length = 800 (ft)
- d. L = 0.13 HRS (hours): from Fig. 7
- e. t_c = 0.22 (hours) = $\frac{5}{3}$ L

VELOCITY METHOD

- *a. land use ______
- *b. slope = ______ (%)
- *c. hydraulic length (HL) = ______ (ft)
- d. velocity (V) = ______ (fps): from Fig. 8
- e. $t_c = \frac{HL}{3600V}$ ______ (hours)

4. Estimate unit peak discharge (q'_p) = 770 ($cfs/mi^2/in$): use Fig. 9
5. Estimate peak discharge $q_p = q'_p AQ$ = 13.9 cfs (cfs)

770 × .0071 × 2.55 = 13.9 cfs.

* indicates required input

TABLE 3. COMPUTATION SHEET: TR-55 GRAPH METHOD – SECTION B

1. Estimate the volume of runoff

   *a. T = 100 (years): return period for design

   *b. P = 4.6 (inches): 24-hr, T-year precipitation volume (i.e., depth)

   *c. CN = 80 : runoff curve number

   d. Q = 2.55 (inches): runoff volume obtained from Eq. 7 or Fig. 5

2. Drainage Area: A = .0116 (Square miles) (7.41 Ac)

3. Estimate Time-of-Concentration (use either the lag method or the velocity method)

LAG METHOD

*a. CN = 80

*b. Slope = 5% (%)

*c. hydraulic length = 1100 (ft)

d. L = .16 (hours): from Fig. 7

e. t_c = 0.27 (hours) = $\frac{5}{3}$ L

VELOCITY METHOD

*a. land use ______

*b. slope = ______ (%)

*c. hydraulic length (HL) = ______ (ft)

d. velocity (V) = ______ (fps): from Fig. 8

e. $t_c = \frac{HL}{3600V}$ ______ (hours)

4. Estimate unit peak discharge (q_p') = 705 ($cfs/mi^2/in$): use Fig. 9

5. Estimate peak discharge $q_p = q_p' AQ$ = 20.8 (cfs)

705(.0116)(2.55) = 20.8 cfs

*indicates required input

TABLE 3. COMPUTATION SHEET: TR-55 GRAPH METHOD - SECTION D

1. Estimate the volume of runoff

   *a. T = 100 (years): return period for design

   *b. P = 4.6 (inches): 24-hr, T-year precipitation volume (i.e., depth)

   *c. CN = 80 : runoff curve number

   d. Q = 2.55 (inches): runoff volume obtained from Eq. 7 or Fig. 5

2. Drainage Area: A = .0016 (Square miles) (1.02 Ac)

3. Estimate Time-of-Concentration (use either the lag method or the velocity method)

   LAG METHOD

   *a. CN = 80

   *b. Slope = 20% (%)

   *c. hydraulic length = 400 (ft)

   d. L = .035 (hours): from Fig. 7

   e. t_c = .058 (hours) $= \frac{5}{3} L$

   VELOCITY METHOD

   *a. land use ______

   *b. slope = ______ (%)

   *c. hydraulic length (HL) = ______ (ft)

   d. velocity (V) = ______ (fps): from Fig. 8

   e. $t_c = \frac{HL}{3600V}$ ______ (hours)

4. Estimate unit peak discharge (q_p') = 1060 (cfs/mi^2/in): use Fig. 9

5. Estimate peak discharge $q_p = q_p' AQ$ = 4.3 (cfs)

$1060\,(.0016)\,2.55 = 4.3$ cfs

*indicates required input

TABLE 3. COMPUTATION SHEET: TR-55 GRAPH METHOD

1. Estimate the volume of runoff
   *a. T = 100 (years): return period for design
   *b. P = 4.6 (inches): 24-hr, T-year precipitation volume (i.e., depth)
   *c. CN = 80 : runoff curve number
   d. Q = 2.55 (inches): runoff volume obtained from Eq. 7 or Fig. 5

2. Drainage Area: A = .0031 (Square miles) (2.00 Ac.)

3. Estimate Time-of-Concentration (use either the lag method or the velocity method)

LAG METHOD

*a. CN = 80
*b. Slope = 20% (%)
*c. hydraulic length = 700 (ft)
d. L = .055 (hours): from Fig. 7
e. t_c = .092 (hours) = $\frac{5}{3}$ L

VELOCITY METHOD

*a. land use ______
*b. slope = ______ (%)
*c. hydraulic length (HL) = ______ (ft)
d. velocity (V) = ______ (fps): from Fig. 8
e. $t_c = \frac{HL}{3600V}$ ______ (hours)

4. Estimate unit peak discharge (q_p') = 1010 (cfs/mi^2/in): use Fig. 9
5. Estimate peak discharge $q_p = q_p' AQ$ = 8 cfs (cfs)

*indicates required input

1010(.0031)(2.55) = 7.98 cfs

TABLE 3. COMPUTATION SHEET: TR-55 GRAPH METHOD

1. Estimate the volume of runoff
   - *a. T = 100 (years): return period for design
   - *b. P = 4.6 (inches): 24-hr, T-year precipitation volume (i.e., depth)
   - *c. CN = 80 : runoff curve number
   - d. Q = 2.55 (inches): runoff volume obtained from Eq. 7 or Fig. 5

2. Drainage Area: A = 0.0012 (Square miles) (0.75 Ac)

3. Estimate Time-of-Concentration (use either the lag method or the velocity method)

LAG METHOD

- *a. CN = 80
- *b. Slope = 4 (%)
- *c. hydraulic length = 400 (ft)
- d. L = .076 (hours): from Fig. 7
- e. t_c = .127 (hours) $= \frac{5}{3} L$

VELOCITY METHOD

- *a. land use ______
- *b. slope = ______ (%)
- *c. hydraulic length (HL) = ______ (ft)
- d. velocity (V) = ______ (fps): from Fig. 8
- e. $t_c = \frac{HL}{3600V}$ ______ (hours)

4. Estimate unit peak discharge (q_p') = 935 ($cfs/mi^2/in$): use Fig. 9
5. Estimate peak discharge $q_p = q_p' AQ$ = 3 (cfs)

---

*indicates required input

935(.0012)(2.55) = 2.86

TABLE 3. COMPUTATION SHEET: TR-55 GRAPH METHOD SECTION 1

1. Estimate the volume of runoff

   *a. T = 100 (years): return period for design

   *b. P = 4.6 (inches): 24-hr, T-year precipitation volume (i.e., depth)

   *c. CN = 80 : runoff curve number

   d. Q = 2.55 (inches): runoff volume obtained from Eq. 7 or Fig. 5

2. Drainage Area: A = 0.0084 (Square miles) (5.40 Acres)

3. Estimate Time-of-Concentration (use either the lag method or the velocity method)

LAG METHOD

*a. CN = 80

*b. Slope = [illegible]% (%)

*c. hydraulic length = 900 (ft)

d. L = 0.15 (hours): from Fig. 7

e. t_c = 0.25 (hours) = $\frac{5}{3}$ L

VELOCITY METHOD

*a. land use ______

*b. slope = ______ (%)

*c. hydraulic length (HL) = ______ (ft)

d. velocity (V) = ______ (fps): from Fig. 8

e. $t_c = \frac{HL}{3600V}$ ______ (hours)

4. Estimate unit peak discharge (q'_p) = 725 (cfs/mi^2/in): use Fig. 9

5. Estimate peak discharge $q_p = q'_p AQ$ = 16 (cfs)

---

*indicates required input   725(.0084)(2.55) = 15.5

TABLE 3. COMPUTATION SHEET: TR-55 GRAPH METHOD Section 7

1. Estimate the volume of runoff

   *a. T = 100 (years): return period for design

   *b. P = 4.6 (inches): 24-hr, T-year precipitation volume (i.e., depth)

   *c. CN = 80 : runoff curve number

   d. Q = 2.55 (inches): runoff volume obtained from Eq. 7 or Fig. 5

2. Drainage Area: A = 0.012 (Square miles) (7.84 ac)

3. Estimate Time-of-Concentration (use either the lag method or the velocity method)

LAG METHOD

*a. CN = 80

*b. Slope = 6 (%)

*c. hydraulic length = 1200 (ft)

d. L = 0.16 (hours): from Fig. 7

e. t_c = 0.27 (hours) = $\frac{5}{3}L$

VELOCITY METHOD

*a. land use ______

*b. slope = ______ (%)

*c. hydraulic length (HL) = ______ (ft)

d. velocity (V) = ______ (fps): from Fig. 8

e. $t_c = \frac{HL}{3600V}$ ______ (hours)

4. Estimate unit peak discharge (q_p') = 700 (cfs/mi^2/in): use Fig. 9

5. Estimate peak discharge $q_p = q_p' AQ$ = 21 (cfs)

*indicates required input

700(.012)(2.55) = 21.4

TABLE 3. COMPUTATION SHEET: TR-55 GRAPH METHOD

1. Estimate the volume of runoff

   *a. T = 100 (years): return period for design

   *b. P = 4.6 (inches): 24-hr, T-year precipitation volume (i.e., depth)

   *c. CN = 80 : runoff curve number

   d. Q = 2.55 (inches): runoff volume obtained from Eq. 7 or Fig. 5

2. Drainage Area: A = .016 (Square miles) (10.26 ACRES)

3. Estimate Time-of-Concentration (use either the lag method or the velocity method)

LAG METHOD

*a. CN = 80

*b. Slope = 6 (%)

*c. hydraulic length = 1700 (ft)

d. L = 0.21 (hours): from Fig. 7

e. t_c = 0.35 (hours) = $\frac{5}{3}$ L

VELOCITY METHOD

*a. land use ______

*b. slope = ______ (%)

*c. hydraulic length (HL) = ______ (ft)

d. velocity (V) = ______ (fps): from Fig. 8

e. $t_c = \frac{HL}{3600V}$ ______ (hours)

4. Estimate unit peak discharge (q_p') = 615 ($cfs/mi^2/in$): use Fig. 9

5. Estimate peak discharge $q_p = q_p' AQ$ = 25 (cfs)

*indicates required input

615(.016)(2.55) = 25.1

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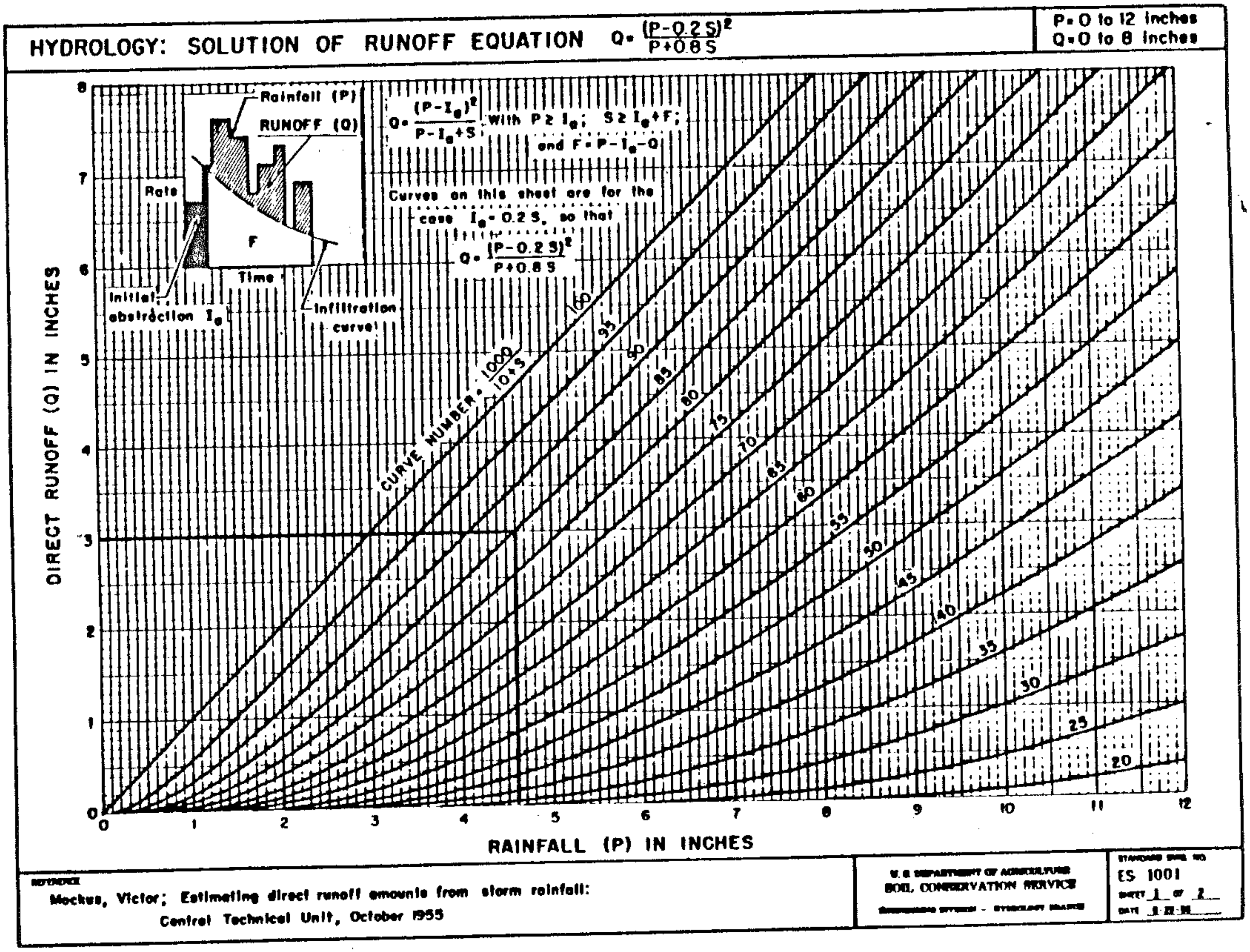

FIGURE 5. Graphical Solution of Rainfall-Runoff Equation

HYDROLOGICAL ANALYSIS

RUNOFF FROM ACTIVE LANDFILL AREA